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Institution

MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS;

WITH OTHER

SELECTED AND ABSTRACTED PAPERS.

VOL. LIV.

SESSION 1877-78.—PART IV.

EDITED BY

JAMES FORREST, Assoc. Inst. C.E., Secretary.

INDEX, PAGE 401.

LONDON:
Published by the Institution,
25, GREAT GEORGE STREET, WESTMINSTER, S.W.
1878.

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LONDON: PRINTED BY WILLIAM CLOWES AND SONS, STAMFORD STREET AND CHARING CROSS.

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ERRATA.

Vol. xlii., p. 236, line 8, for "10·7" read "20·7."
" " line 10, for "87·0" read "97·0."
Vol. liii., p. 2, line 3, for "Dovey" read "Dorey."
" p. 255, line 5, for "$n\ d_s\ t$ cosec θ" read "$n\ d_t\ t$ cosec θ."
" p. 256, line 12 from bottom, for "V" read "V_o."
" " line 4 from bottom, for "h_2" read "h_t."
" p. 259, line 16, for "v_t^2" read "V_t^2."
" p. 262, line 2, for "u_s" read "v_s."
" p. 265, line 6, for "ϕ cosec^2" read "cosec$^2\ \phi$."
" " line 6 from bottom, for "$u_t \cot^2 \theta$" read "$u_t^2 \cot^2 \theta$."
" p. 270, line 3, for "$v_t\ u_t \cot \theta$" read "$V_t\ u_t \cot \theta$."
" " line 10, omit this line.
" " line 14, for "v_o" read "V_o."
" p. 278, equations 25 and 26 for "$2\ k\ V_o\ u_o \cot \phi$" read "$(2 - 2\ k^2)\ V_o\ u_o \cot \phi$."
" " line 16, for "$- 6\ v_o\ u_o \cot \phi$" read "$+ 6\ v_o\ u_o \cot \phi$."
" p. 319, line 16 from bottom, for "5½ kilogrammes" read "5½ kilomètres."
Vol. liv., p. 291, in Table 2, under Remarks, line 3 from top, for "sheet" read "short."

THE

INSTITUTION

OF

CIVIL ENGINEERS.

SESSION 1877-78.—PART IV.

SECT. I.—MINUTES OF PROCEEDINGS.

April 9, 1878.

JOHN FREDERIC BATEMAN, F.R.SS. L. and E., President, in the Chair.

No. 1,561.—"The Victoria, Albert, and Chelsea Embankments of the River Thames." By EDWARD BAZALGETTE, Assoc. Inst. C.E.[1]

THE river Thames is the arterial drain of about 5,264 square miles of country. It supplies London with water, and forms the great highway for the commerce of that city to and from all foreign ports. At its source, near Selbury in Gloucestershire, it is 330 feet above the mean level of the sea; from thence to its mouth the river traverses a distance of 210 miles. Up to Teddington Lock, nearly one-fourth of the whole length, it is tidal; the discharge of water at Teddington in dry weather is about 470,000,000 gallons per diem. The tidal waters in former years overflowed large tracts of land in Kent and Essex; but with the development of civilization its waters were restrained by embankments, which were formed and have been upheld under the authority of various Acts of Parliament. The navigable channel up to London has thus been improved and maintained, and the original waste and unhealthy marshes have been converted into fertile and inhabited districts. There is every indication that one branch of the Thames formerly flowed from Greenwich Reach through Lambeth to Lambeth Reach in the line of the Grand Surrey Canal, and another from Blackwall Reach through the present West India Docks to Limehouse Reach. Nor can it be

[1] The discussion upon this Paper occupied portions of two evenings, but an abstract of the whole is given consecutively.

doubted that the Strand, since raised by artificial deposits to considerable elevation above the bed of the river, derived its nan from having been literally the strand or foreshore of the river be Thus the river, formerly divided into several channels within tl metropolis, has been confined within one deeper channel of a mo uniform section.

By the removal of Old London, Westminster, and Blackfria bridges, and by the substitution of bridges of wider spans wi narrower piers and deeper foundations, the tide is enabled flow and ebb more freely, which has deepened the navigal channel. The width of the river, however, is still very varial (Plate 1). Above Southwark Bridge it is only 670 feet from ba to bank, and the waterway between the piers of that bridge is ab 600 feet. At Hungerford Bridge (the site of the present Char Cross Bridge), before the formation of the Thames Embankme the width from bank to bank was 1,340 feet; whilst oppo Millbank it was only 610 feet broad, again widening to nea double that breadth at Battersea. Mud banks were frequent the foreshore and in the river, and upon the north bank, betw Westminster and Blackfriars bridges, about 27 acres of mud b were exposed at low water.

The first authentic information respecting the embankment improvement of the Thames commences in the year 1367, in reign of Edward the Second, when certain persons were constitu Commissioners, by King's letters patent, to view from time time and repair the river banks. The first Acts of Parliam for the preservation of the river banks and for facilitating n gation, were passed in the reigns of Henry the Eighth and Elizabeth. Sir Christopher Wren, in preparing his design rebuilding London after the great fire in 1666, appears to h conceived the idea of reclaiming from the river a portion of mud banks, and of improving it by the formation of an embankm on the Middlesex shore from the Temple Gardens to the Tow and in 1668 an Act was passed to prevent the erection of buildi within 40 feet of the river bank between the Tower and Temple. A similar project was advocated by Sir Frederic Tre and Mr. Martin, the painter, rather more than forty years a the latter combining with his design a scheme for the int ception of the sewage from the river. Mr. James Walker, Pa President, Inst. C.E., in 1840 laid down a line, for the Corporati of London, as that to which wharves or embankments betwe Westminster and Blackfriars bridges might be extended into th river. This line with but slight modifications has since bee

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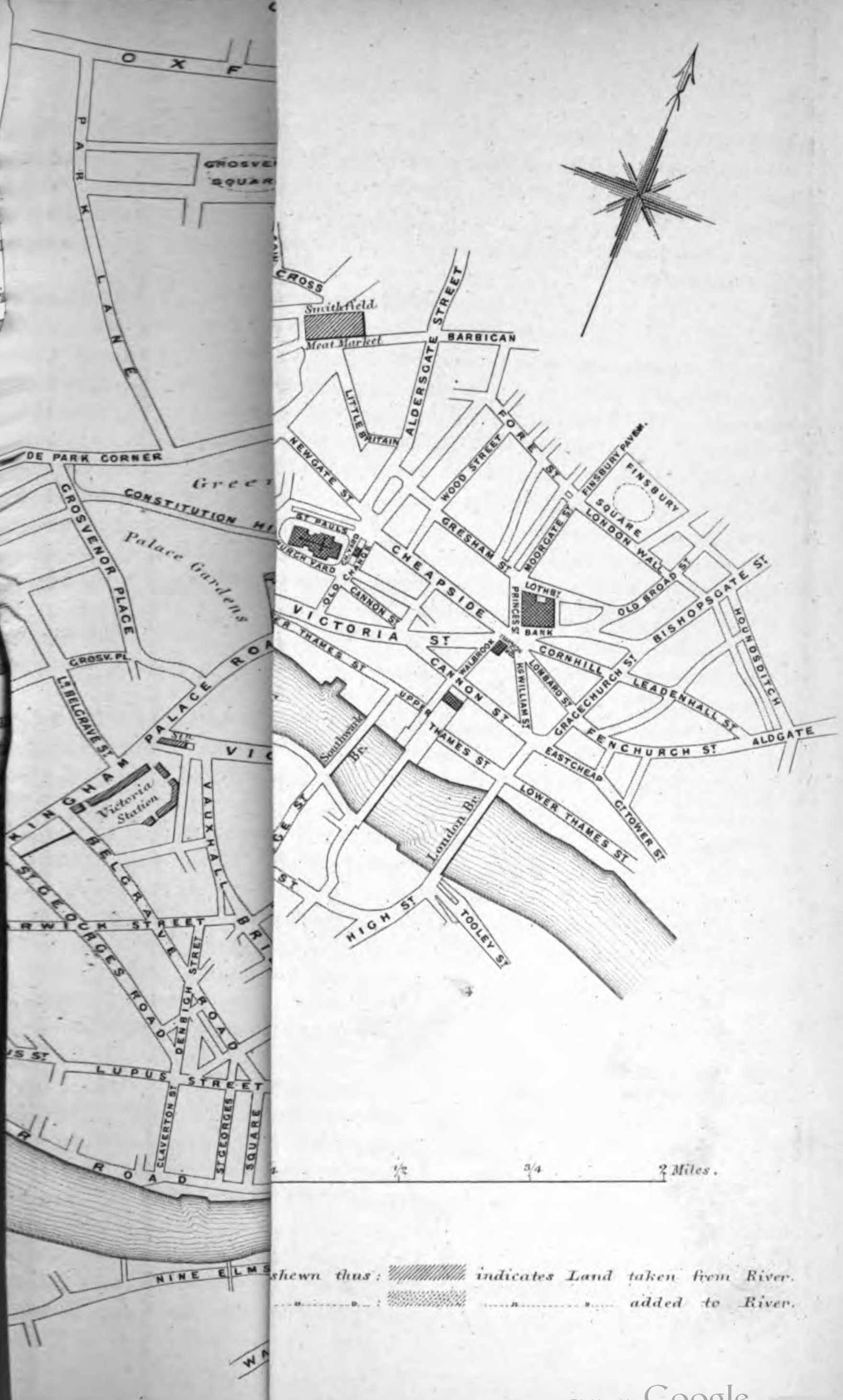

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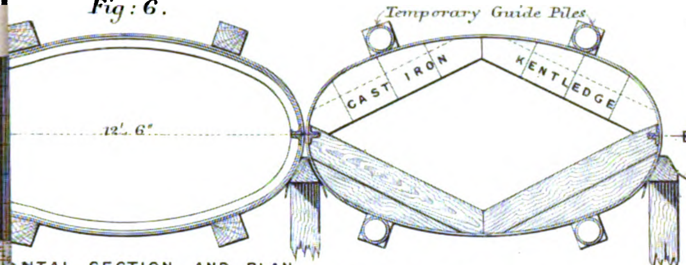
Fig: 6.



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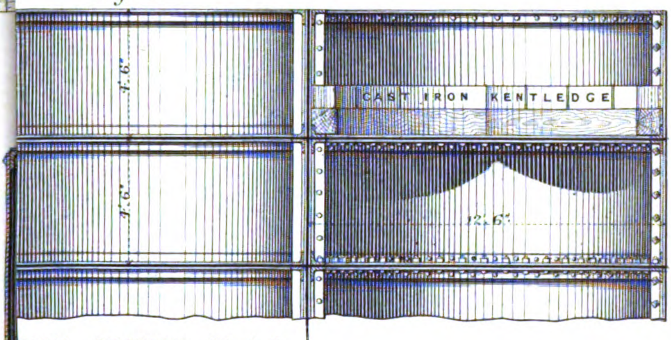
…and partly round the Timbering indicate the tarred Felt Packing.—

Fig: 7.



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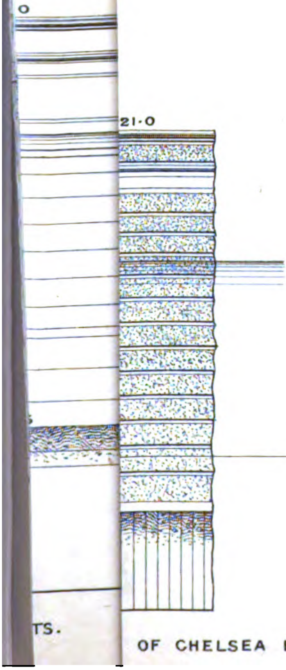
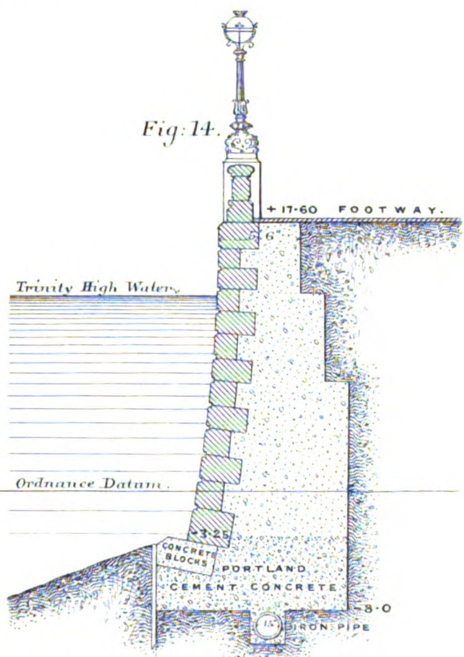


Fig: 14.



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…OF CHELSEA EMBANKMENT.

adopted; and the execution of the embankment, so frequently mooted and so long delayed, was eventually brought about as an adjunct to the drainage works of the metropolis.

Up to 1815 all the sewage of London was retained in cesspools, and it was penal to allow it to flow into sewers; but in 1847 an Act was passed which made it compulsory to drain the sewage into sewers. Within six years after the passing of that Act, thirty thousand cesspools had been abolished, and the drainage of the houses turned into the Thames. As the population of London increased and the drainage improved, the quantity of sewage discharged into the river became proportionately greater, until its mud banks were so polluted with sewage and its waters so offensive as to cause serious alarm. In 1856, an Act was passed authorizing the diversion from the river within the metropolis of all sewage, by the formation of intercepting sewers. Great difficulty and inconvenience to the public were anticipated in forming a sewer along the Strand. It was considered that not only would the difficulty be removed by the formation of an embankment on the Middlesex shore, but that the channel of the river would be improved, the mud banks being converted into valuable and ornamental lands, and a new line of thoroughfare obtained. Such considerations induced the Metropolitan Board of Works to urge upon the Government the investigation of these questions. Thus, various Parliamentary Committees and Royal Commissions deliberated and reported upon the plans submitted to them by Messrs. Fowler, M'Clean, Hemans, Page, Shields, Gisborne, Bazalgette, and others.

Although mud banks of considerable extent exist upon the Surrey side, between Westminster and Blackfriars, and the embankment of that shore would much improve the appearance of the river, yet the advantages to be obtained by such a work would not be equal to those on the north side; for no public roadway is needed there, nor any sewer, and the claims for compensation to the owners of the wharves would be heavy. The south side of the river, from Westminster to near Vauxhall, has, however, been embanked under an Act obtained by the Metropolitan Board of Works in 1863, and a further embankment of the river on the north side, from Chelsea Hospital to Battersea, has also been completed, an Act for the same having been obtained in 1868.

All these works have been executed according to the designs and under the superintendence of Sir Joseph Bazalgette. It is proposed in this Paper to give a more detailed description of the mode of constructing the Victoria Embankment, as it was the first, the

largest, and the most difficult work; and afterwards to describe briefly the main features, and to allude only to the chief points of difference in the modes of construction, of the other embankments. The three works comprise about 3½ miles of embankment and public roadway, and have reclaimed about 52 acres of river mud.

The Victoria Embankment (Plates 1 and 2).

The Victoria Embankment, between Blackfriars and Westminster bridges, was commenced in 1864 and was opened in 1870. The Act for its execution was obtained under the direction of the First Commissioner of Her Majesty's Works in 1862, in pursuance of a report of a Royal Commission of the previous year, and the work was confided to the Metropolitan Board of Works. By this Act an embankment, corresponding to Mr. Walker's lines, was laid down, and the designs were subsequently prepared by Sir Joseph Bazalgette, and executed under his superintendence, with the assistance of Messrs. Lovick and Cooper, the resident engineers. The curvature of the Embankment is as follows:—Between Westminster Pier and Hungerford (Charing Cross) Bridge the curve is compound, being set out to radii of 8,800 and 1,850 feet; between Hungerford (Charing Cross) and Waterloo Bridges the curve is of 1,655-feet radius, and below Waterloo, extending to 100 feet beyond the Temple Pier, the radius is 8,520 feet. The abutments and piers of the several bridges which meet the curved line of wall are tangential to the curve.

This embankment was divided into three contracts. Mr. Furness executed the portion between Westminster and Waterloo bridges; Mr. Ritson, the contract from Waterloo Bridge to the eastern end of the Temple, and Mr. Webster the one from the Temple to Blackfriars Bridge, and the formation of the whole of the roadway. The total estimated cost of the work was £1,200,000, and a further sum of £450,000 was paid for the purchase of property and for compensation.

The roadway from Westminster to Blackfriars is 100 feet wide and about 1¼ mile in length. It has been extended from Blackfriars Bridge to the Mansion House, a further length of ¾ mile, of a width of 70 feet, with the exception of a short length of the present New Earl Street, which is only 60 feet wide. The 70-feet roadway is divided into a carriageway of 46 feet and two footways of 12 feet each. The roadway between the Mansion House and New Earl Street was opened to the public in October 1869. The total area of the land thus reclaimed from the river

was $37\frac{1}{4}$ acres, of which 19 acres are occupied by carriage and footways; $7\frac{1}{2}$ acres have, under the Act of Parliament, been conveyed to the Crown, the Societies of the Inner Temple and Middle Temple and adjacent landowners; and about 8 acres have been devoted to the use of the public as ornamental grounds. The Templars are restricted from building over their portion of reclaimed land. The main roadway is divided into a central carriageway 64 feet in width, with two footways, that on the land side being 16 feet wide and that on the river side 20 feet, along which are planted rows of plane trees at intervals of 20 feet apart. The public way is protected on the river side by a moulded granite parapet 3 feet 6 inches in height; on the land side it is divided from the garden grounds by an ornamental cast-iron railing. Opposite Whitehall Gardens the separation has been effected by a wall of masonry and brickwork 7 feet 6 inches high, and from the Temple Gardens to Chatham Place by a brick parapet about 5 feet 6 inches in height.

The Act of 1862 contemplated a solid embankment and roadway 100 feet in width, as far eastward only as the end of the Temple Gardens, the roadway thence to Chatham Place being reduced to 70 feet in width and carried on arches, under which the river could flow to existing wharves. Subsequently, however, the Metropolitan Board of Works, in conjunction with the Metropolitan District Railway Company, obtained power to form the roadway of the full width of 100 feet, and on a solid embankment all the way to Blackfriars Bridge. The footways are paved with 3-inch York paving, with granite kerbs, and the carriageway is macadamised.

A double covered way has been carried under the roadway opposite the site of the City Gas Works, with a landing wharf on the river front, by means of which the Gas Company is enabled to convey coals from the river to their premises without interference with the public roadway.

The approaches to the Embankment are from Charing Cross, from Westminster and Blackfriars bridges, and from Whitehall Place, Villiers, Norfolk, Surrey, and Arundel streets. An approach on a viaduct from Lancaster Place, Waterloo Bridge, passing through the ornamental grounds in front of the Adelphi, was provided in the original Act, but has since been abandoned. One from Charing Cross to the Embankment, contemplated in the original design, was also abandoned on account of the opposition of the Duke of Northumberland. Eventually the Duke consented to convey his property to the Metropolitan Board of Works for the formation

of the new roadway, and the approach known as Northumberland Avenue has since been formed.

The level of the Embankment main roadway is about 4 feet above Trinity high water, except towards Westminster and Blackfriars bridges, where it rises to a height of about 20 feet above high water. The rising ground for both these approaches is retained, on the river side, by a granite-faced wall similar in character to the general Embankment wall. The parapet has a moulded cap and base, and is 3 feet 6 inches above the footpath, making the total height of the Embankment wall 40 feet. The rising ground at Blackfriars Bridge is supported on the land side by a retaining wall of concrete, the wall being carried on piers and arches of concrete at a level of 10 feet below datum, and surmounted with a parapet of brickwork in mortar, capped with Portland stone.

A height of 4 feet above Trinity high water, or 16 feet 6 inches above ordnance datum, had been laid down by the late Mr. James Walker, in his official reports upon the embankments of the Thames, as that to which no tide, up to that period, had ever risen. Of late years there have been frequent and serious overflows of the banks of the river. On the 15th of November, 1875, the tide attained the unprecedented height of 17 feet 1 inch above ordnance datum, and in January, 1877, it reached the height of 16 feet 6 inches above ordnance datum. A popular theory, that this increased rise of tide is due to the narrowing of the river by the formation of the new Thames embankments, has been controverted in the evidence of several engineers adduced before the Committee of the House of Commons of last Session on the flooding of the river Thames. The Author refers particularly to the evidence of Mr. Law, as to the changes which have taken place, and their effect upon the tidal movement. He submits that whilst it is undoubtedly the fact, that the tide now rises to a greater height than formerly, the increased altitude is not due to the Thames Embankment, but must be attributed to the removal of Old London, Blackfriars, and Westminster bridges and of other obstructions, and to the deepening of the bed of the river, and consequent increased volume of tidal water which flows through London. These phenomena, although incidental only to the subject of this Paper, are of the utmost importance to London, and to all towns situated upon tidal rivers.

Within the Embankment wall, and forming a portion of its structure, is the Low Level Intercepting Sewer, and above it is a subway for gas and water pipes. The subway is 7 feet 6 inches in height, and 9 feet in width, and the sewer varies from 7 feet

9 inches to 8 feet 3 inches in diameter. Both are situate under the footway next the river, and form a buttress to the wall.

The Embankment wall (Plate 2, Figs. 2 and 11) from the roadway level to the toe of the brickwork facing a portion of it, presents a slightly curved batter. From the base to mean high-water level it is plain; above that level it is ornamented with mouldings, which are stopped at intervals of about 70 feet with plain blocks of granite, serving as lamp pedestals, and relieved on the river face with bronzed lions' heads, carrying mooring rings. The uniform line of embankment is broken at intervals by massive piers of granite, flanking recesses for pontoons, or landing stages for steamboats, and at other places by projecting stairs, intended as landing stages for small craft. The steamboat piers occur at Westminster, Charing Cross, and Waterloo bridges; those for small boats are formed midway between Westminster and Charing Cross, and between Charing Cross and Waterloo bridges, and landing places for both are combined opposite Essex Street. It is intended, eventually, to surmount the blocks and pedestals with groups of statuary. The obelisk called Cleopatra's Needle is about to be erected on the landing place midway between Charing Cross and Waterloo bridges.

The landing stages for steamboats are formed of timber platforms carried upon wrought-iron barges, for which purpose the caissons used in the formation of the iron cofferdams have been employed. These platforms, which rise and fall with the tide within recesses formed by projecting granite piers, support the lower ends of bridges or gangways, the upper ends of which are hinged to the masonry at the level of the public footpath, and thus form inclined planes of constantly varying gradient for the approach of passengers from the roadway to the steamboats.

The bridges are composed of two wrought-iron girders carrying a timber flooring between them. They rise and fall within two granite walls parallel to the general line of embankment. Upon the platforms are erected waiting-rooms and other accommodation for public convenience.

In connection with the steamboat pier at Westminster Bridge there is a subway under the roadway to communicate with the subway previously formed under Bridge Street. These subways afford an underground thoroughfare for foot-passengers between the Houses of Parliament, the Metropolitan District railway station, the steamboat pier, the footways in Bridge Street, and those on the Embankment.

The Metropolitan District railway enters the lands reclaimed by

the Embankment near Westminster Bridge, and passes under the public road as far as Charing Cross steamboat pier. Here it diverges under private ground on the land side of the roadway to the Charing Cross station, the roof of which rises above the surface. Immediately east of the station are three openings for ventilation, which, together with the screen wall, are partially concealed by the mounds and shrubberies of the ornamental grounds. East of the openings the railway is carried in a covered way under the ornamental grounds as far as the Waterloo steamboat pier, when it again passes under the roadway to the Temple Station, and is thence continued on the land side of the roadway to within a few feet of Blackfriars Bridge. From the east end of the Temple Gardens the concrete wall, which retains the earth for the rising approach-road to Chatham Place, forms also the side walls of the railway. The level of the rails is about 17½ feet below the surface of the roadway, which is carried by cast-iron girders and brick arches, the under sides of which are 18 inches below the surface of the road.

The Embankment wall (Plate 2, Fig. 2) is constructed of brickwork faced with granite, and has been carried to a depth of 32½ feet below Trinity high-water mark, and 14 feet below low water, the foundation being of Portland cement concrete. The execution of works under such circumstances necessitated their formation within cofferdams, and involved many difficulties, especially where bad ground was encountered, or where the excavations had to be carried on in proximity to the bridges and other heavy buildings. In all cases whole-tide dams were driven into the clay, mostly formed of two rows of timber piles with puddle filling between, as described by Mr. Ridley, Assoc. Inst. C.E.[1] The remaining lengths were formed of wrought-iron caissons sunk into the bed of the river.

To determine the nature of the sub-strata preliminary to designing the work, fourteen bore-holes were sunk along the intended line of wall, seven between Westminster and Waterloo bridges, and seven between Waterloo and Blackfriars bridges. The results of these borings showed the average depth of the surface of the clay between Westminster and Waterloo to be about 28 feet below ordnance datum, although great irregularity existed (Plate 2, Figs. 3, 4 and 5); for instance, between York Gate and Charing Cross Bridge, borings Nos. 3 and 4 showed the clay level to vary respectively between − 33·15 feet and − 17·09 feet, or a difference of level of about 16 feet, while near Westminster Bridge it was

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxxi., p. 3.

at — 31·96 feet. The seven borings between Waterloo and Blackfriars exhibited greater uniformity in the depth at which the clayey substratum occurred; its average depth was shown to be — 30 feet, although subsequent operations proved its general level between these lines to be — 25 feet. Previous to commencing the work, sixty-one cross sections were taken at equal intervals along the old shore walls, extending from thence towards the river to a little beyond the present line of wall. These cross sections showed that the foreshore had a total fall of 13 feet in the reclaimed land. The mud within this area was removed by barges previous to the whole space being covered with ballast, or other approved filling, which was then brought up from the more solid surface beneath the muddy foreshore to levels varying from 15 feet above ordnance datum, for a width of 120 feet at the back of the new wall, and from 15 to 13½ feet above this datum for the remaining width, to the original existing river walls or banks.

After the formation of the dams, and when the excavations had been sunk to the depth of 20 feet below datum, the trenches were filled with layers of concrete to a level of 12½ feet below ordnance datum, at which height a bed was formed to receive the footings of the brick wall. All the Portland cement was subjected to a rigid test, and was rejected when not found capable of resisting a tensile strain of 250 lbs. per square inch, after seven days' immersion in water. The concrete formed of this cement was used in the construction of the river wall in the proportions of 1 of Portland cement to 6 of ballast, up to a level of 8 feet below datum, and above this level the proportions of cement and ballast were as 1 to 8.

The brickwork was laid in courses at right angles to the face of the wall, and was thoroughly bonded with the stone facing, being brought up simultaneously with it. The low level sewer was supported on concrete foundations (Plate 2, Fig. 2), subject, however, to an increase or diminution of depth, according to the nature of the soil. The sewer was of brickwork 13½ inches thick, and was incorporated with the concrete and wall. When the wall and concrete covering the sewer were raised to a level of about 6·87 feet above datum, the brick subway was formed over the low level sewer; its side walls being 1 foot 6 inches, and the brick arch 13½ inches thick. The subway was generally constructed upon a level, excepting where it crosses the sewer storm overflow chambers, and at or near Waterloo and Blackfriars bridges; and in certain places its floor was constructed with a brick invert instead of concrete.

The subway, sewer, and river wall have been tied into each

other at intervals of 6 feet by cross walls 18 inches in thickness. These cross walls extend from the brickwork of the river wall to a vertical line 9 inches beyond the side of the sewer furthest from the wall, and from footings 9 feet below datum, bedded on a concrete foundation 12 inches thick, up to the under side of the subway, to a level of 7 feet above datum. The space between the subway and river wall is filled with brickwork. A counterfort of brickwork is built over and around the arch of the subway to a level of 16 feet above datum at the back of the lamp pedestals, to receive a washer plate for attaching the mooring rings, which are thus connected from the front to the back of the wall by rectangular mooring bars 2½ inches thick by ⅝ inch wide. The arch of the subway is in all cases coated on its exterior surface with asphalt.

The stones facing the granite wall are fine axed on the river front to a curved batter of 100-feet radius, struck from a tangent line 16 feet 6 inches above ordnance datum, on a level with the footway; their backs are scabbled to a fair surface to join the brick-backing. From 11½ feet above ordnance datum the mouldings and all exposed surfaces are chisel-dressed to a smooth surface. The stones are all bedded and jointed in neat Portland cement, no joint over ⅛ inch in thickness being permitted.

Where caisson cofferdams were employed, they were left in permanently up to the levels of 10 or 14 feet below ordnance datum; and the timber cofferdams were, in many cases, cut off to about the same level.

The works of the Westminster, Waterloo, and Temple steamboat piers were of a different character from those along the general line of wall. Taking Westminster as an example, the recess formed in the line of this Embankment wall is 250 feet in length. It recedes landwards from the face of the wall to an extent of 29 feet for a length of 90 feet in the centre, and to an extent of 18 feet for a length of 80 feet on each side. The foreshore over the area of the recess where the pontoons float, and in front of the entire pier and of Westminster Bridge, was excavated to 14 feet below ordnance datum; and the foundations of the Embankment wall, and of other walls of this structure, were carried down in brickwork and masonry to a depth of 15 feet 3 inches below datum, and rested on a bed of Portland cement concrete, brought up from a level of more than 20 feet below ordnance datum.

Three flights of steps descend from Westminster Bridge from a level of 30 feet above ordnance datum to the Embankment footway, which is 16 feet 6 inches above datum. In the space beneath the steps a reservoir has been constructed for flushing away any

accumulation of mud within the recess formed for the steamboat pier. This reservoir is constructed of brick arches, 1 foot 10½ inches thick, carried on piers 4 feet 6 inches thick. It is pierced with arched openings, and floored with inverted brick arches 1 foot 10½ inches in thickness, which also spring from the same piers as the covering arches, and rest upon a bed of concrete 1 foot 6 inches thick. Above this reservoir the steps are carried upon three spandrel or groin walls, 1 foot 6 inches thick, also by a dwarf wall upon one arch formed over the subway, all the spandrels being filled with gravel. The reservoir beneath the steps is made available for scouring the floor of the recess by a culvert 4 feet in diameter, built under the low level sewer and the subway, and connected at one end with the reservoir, and at several points along its length with iron pipes having outlets, each of which is fitted with a boxed sluice valve for retaining or discharging the tidal water from the reservoir. The outlets from the sluice valve are flattened, being 4 feet 9 inches wide by 9 inches high, and discharge over the floor of the recess. This floor is composed of square granite blocks, 12 inches wide, 12 inches deep, and 4 feet long, bedded and jointed in Portland cement upon concrete 18 inches thick, laid to a slope, riverwards, of 1 in 15 and 1 in 20. The toe of this apron rests upon a dwarf brick wall with footings, and is further secured by a mass of concrete in front.

In forming the steamboat pier near Waterloo Bridge, the ground was excavated, and the foundations of brickwork and concrete were put in to nearly similar depths as at the Westminster steamboat pier, but the flushing arrangements were different. Instead of a reservoir as at Westminster, a brick culvert, 7 feet wide by 8 feet deep, is carried between the Embankment and the screen walls, at an invert level of 11 feet below datum from end to end of the recesses, and passes at the back of the pier of Waterloo Bridge. Fourteen outlets are formed in the screen wall of each recess, to admit the free passage of tidal water. Two other culverts, each 3 feet wide by 4 feet deep, were also made through each of the piers which form the recesses for the pontoons. The screen walls are tied to the Embankment wall at the back by the culvert, and by walls, 18 inches thick, at intervals of 6 feet 3 inches, the spaces between being arched over in brickwork. As the foundations for this portion of the work were carried down below the footings of the pier of Waterloo Bridge, it was proposed previously to secure the bridge in the following manner:—The ground was to be first excavated around the pier to a depth of 12 feet below

ordnance datum, and the foundations to be completely inclosed by a permanent dam of piles and plates. The former, circular cast-iron screw piles, with a groove on each side, were to be screwed down at intervals of 5 feet, 5 feet $1\frac{1}{2}$ inch, and 4 feet 6 inches from centre to centre, to a depth of 32 feet below ordnance datum. Cast plates, measuring 1 inch thick, with three ribs on the back, each 3 inches deep, and with two ribs on the face, each 2 inches deep, having on the tops an iron skew back to receive a granite apron, were driven in between the grooves of the screw piles to a depth of 22 feet below ordnance datum. The main piles were to be capped with cast iron, in such a manner that the tops of the caps when fixed should be level with the tops of the plates. They were then to be secured by wrought-iron tie rods, $1\frac{1}{4}$ inch in diameter, passed through the pile and pile caps, and screwed at the further extremity to the masonry forming the footings of the bridge pier by wrought-iron lewises 15 inches deep, and tightened up by a nut screwed in front of the pile cap. The material inclosed was finally excavated from the level of $-$ 12 feet to the level of the lowest course of footings of the bridge pier immediately adjacent, and the vacant space was filled with concrete to form a bed for a granite apron in front of the pier. Similar piles and plates were also to be driven at similar depths to form the foundation for a wall built in continuation of the river side of the bridge pier, making a total length of 112 feet 9 inches, inclusive of the present length of pier; this wall was returned at its extremities to meet the Embankment at the back of the recess, of which it forms a side. This work was modified to some extent during its construction.

The substitution of wrought-iron caissons for ordinary timber dams was suggested by Sir Joseph Bazalgette to effect a saving of cost, by using the same caissons two or three times in different parts of the dam. Mr. Furness at once fell in with the view, and spared no effort in assisting to develop the best mode of executing it. The caissons (Plate 2, Figs. 6, 7, 8, 9, and 10), were constructed in wrought iron of half oval rings, with upright flanges at each end, so that when the halves were bolted together they formed a caisson 12 feet 6 inches long by 7 feet wide in the centre and 4 feet 6 inches deep. The plates were $\frac{1}{2}$ and $\frac{3}{4}$ inch thick. Angle-irons were bolted round the top of the rings, enabling the rings to be firmly secured to each other in a vertical position. In every case the lowest ring of each separate caisson was of cast-iron weighing about 32 cwt., with a cutting edge at its lower extremity, so as more readily to penetrate the soil.

In order to form a water-tight joint between the caissons at their junctions, a cast-iron groove, 5¼ inches deep and 6¼ inches wide, was bolted to the flanges at the end of each caisson, so that when the several portions were placed in position, each groove came opposite to that of the contiguous caisson, and they thus formed an aperture 10½ inches long by 6¼ inches wide from top to bottom of the caisson. A half-pile timber of dimensions corresponding to this aperture, was previously driven and acted as a guide, which, with the assistance of a few surrounding piles, maintained the caissons rigid and vertical during their descent; if the joint leaked the wood swelled and prevented any serious flow of water. The cast-iron grooves were, however, too costly and were abandoned, the caissons being then held together longitudinally by bolts passed through the angle-irons, and they were felted at the junctions. Four guide piles were in this case ordinarily driven so as to prevent the caissons swerving from the vertical line during their descent, and thus facilitated the operation of sinking. Besides the guide piles, upright pieces of timber, cut to the shape of the caisson junction, and lined between with felt, were placed on the inner side of the joints, and the timber struts were used for strengthening the dam abutting against them. Thus, in proportion as the pressure of water outside the dam increased the strain upon the struts, the joint became tightened. Half-caisson rings only were used in the upper part of the dam, the convex side facing the river; felted timber and struts supported the end flanges on the inner side of the dam. A frame of timber also stiffened their centre. The application of half-caisson rings in this form effected a considerable reduction in the cost. From Westminster Bridge to 300 feet east of Waterloo Bridge the total number of these caissons was one hundred and ninety-five.

The return dams were in some instances formed of cast-iron boxes fixed upon and parallel to each other. The boxes or coffers consisted of two iron castings, each being double T shape and so united at their extremities as to form a series of pockets 6 feet long by 3 feet wide and 4 feet deep. The dam was gradually raised to the full height by passing bolts through angle-irons secured to the upper and lower extremities of each separate coffer. As the construction of the dam progressed, all the intervening spaces thus formed were filled with clay puddle. This particular form of dam was not extensively adopted, as it lacked rigidity and exhibited signs of failure when subjected to serious hydrostatic pressure. Each of the elliptical wrought-iron caisson rings weighed about 30 cwt., and was supplied at an average cost of £16 15*s.* per ton.

The caissons were sunk by excavating the ground within them, and were weighted with iron blocks of about 9 cwt. each, cast to the shape of the rings, and piled upon timbers which rested upon the flanges and formed the strengthening struts. They were thus lowered 4 feet or more into the London clay, which between Westminster and Hungerford Bridges was found to vary between − 33·15 feet and −·17·09 feet below ordnance datum. In every caisson a ring, provided with one or more sluices, was fixed a little above low-water level to discharge water from the interior, and, if necessary, from the enclosed space behind the dam; also in an emergency to admit the tidal water. These sluices were connected with iron rods worked by a lever from the top of the dam. Previous to putting in the foundation for the Embankment wall, the caisson rings were sunk at various depths into the London clay, and the lowest portion was filled with Portland cement concrete up to a level of about 14 feet, or 8 feet below low water, and so became part of the structure. Thus the concrete at the toe of the wall, together with the gravelly sub-stratum upon which it rested, was completely enclosed, and a firm and deep foundation economically obtained.

The method adopted for getting in the foundations of the river wall, where the iron caisson dams were used, was somewhat similar to that employed for the wooden cofferdams. The site upon which any given length of wall was about to be constructed was first sufficiently surrounded with caissons, sunk to the full depth, and these were filled with concrete up to the level intended to remain as permanent work. The return dams were carried into the earth at the back of the dam, so as to exclude the river water. Four rows of parallel waling (Plate 2, Fig. 1), were fixed against the inner sides of the caissons. The position of the upper row was about 2 feet below Trinity high water and that of the lower one near low-water level. Against these walings abutted horizontal struts which thrust against upright piles 12 feet apart in the length of the dam and driven 6 feet into the London clay. These piles were supported by four struts raking in such a way as to transmit the pressure at the walings against two other piles driven 64 feet from the inner side of the dam and backed with a 10-feet width of 6-inch deals, supported in the rear by approved filling.

In every cofferdam one or more of the caissons was provided with sluices fitted with penstocks. These passed through their interior a little above low-water level, so that, if necessary, the tide could be allowed to ebb and flow within the interior of the cofferdam.

A sump was also sunk several feet into the clay within the cofferdam, and open-jointed pipes, laid throughout the length of the enclosed area, were connected with it. The trench was then excavated to the dimensions necessary for the construction of the foundations of the river wall. The land water, which rapidly accumulated in the sump, was raised by chain and bucket pumps and thence conveyed into the river by wooden shoots. In some cases the water arrived in such volumes as to overpower the pumps, and necessitated putting in the concrete through several feet depth of water. This operation was, however, performed without difficulty or detriment to the concrete. Boxes of iron and wood were provided with arms which worked as scissors and opened and closed the box: into these the concrete was filled and lowered through the water. On reaching the bottom of the trench they were opened and the concrete deposited.

The gravel was in many instances excavated by 8-HP. steam dredgers, supported upon a 55-feet traveller, along which rails were laid. The traveller stretched across the width of the trench. It worked upon a gantry formed of rails bolted into walings, which were carried upon the tops of piles driven into the clay at equal distances apart along the length of the dam. The whole of the enclosed area was thus commanded by the combined action of the dredger and the traveller. About four travellers were employed for the caisson dams. These were used in raising and lowering the caissons, as well as in aiding the removal of the dams.

As soon as the wall had been brought up a few feet above low-water level, the sluices were generally capable, when the tide had sufficiently lowered, to discharge the water accumulated within the dam during the rise and fall of the tide, so that the remainder of the work could be got in without pumping. The timbers for strengthening were removed from across the line of wall as the work progressed, and the dams were supported instead by short struts, which abutted against the face of the wall.

The three principal modes for excavating the materials within the cylinders were as follow :—

1. By men inside, the water being kept down by a chain and bucket pump.

2. By men inside, the water being kept out by pneumatic pressure.

3. By a telescopic dredger, worked with an endless chain and buckets, the water being allowed to rise and fall within the cylinders.

By the first mode the soil was filled into and raised in skips,

during which process a chain and bucket pump, constantly at work in a sump within the caissons, discharged the water by a wooden shoot into the river. The average amount of soil excavated in this way amounted to 6 cubic yards per diem. In the pneumatic system a plate of wrought iron, with a man-hole cut in it, was firmly bolted to the upper end of a caisson ring, intended to form the covering of the caisson; and an iron chamber, of sufficient capacity to hold one man, was fastened immediately under the man-hole. It was likewise furnished with a man-hole flap valve, which led into the interior of the caisson. Each man-hole was fitted with flaps or doors, worked by equalising the pressure between the caisson and chamber. For this purpose the iron chamber was also provided with two pipes, one communicating with the interior of the caisson, the other with the atmosphere outside. Air was pumped, with ordinary apparatus, into a hole passing through the covering of the caisson. The pressure of air within thus closed the lower door or valve, so that a man could freely enter the chamber through the upper man-hole, and, after closing the door, he could so equalise the pressure within the caisson and chamber as to pass into the caisson through the lower door, and thereby descend by a ladder. The soil excavated was filled into buckets or small skips, and raised by a wire rope, which passed through a stuffing-box constructed in the caisson cover. On drawing up a full bucket to the level of the lower door of the chamber, the pressure of air was again equalised by a man who constantly worked within it, and the door was opened so that the bucket could be drawn in by him and again be passed out through the upper man-hole. The amount of earth excavated by this method was about 5·31 cubic yards per diem. The telescope dredger proved useful for every description of excavation under water, and was applied with great advantage in sinking the caissons, as well as in excavating the trench for the foundations of the Embankment. It consisted of a pair of iron frames, which were secured to timbers fixed above the uppermost caisson ring, the dredger being worked by a strap placed at a convenient distance from the caisson. The sword, or ladder, carrying the bottom cam was made of two channel irons tied together by eight lattice braces, having their sides so slotted as to form toothed racks, into which geared two pinions. The shaft carrying the pinions was furnished with double-acting ratchet wheels and pawls, which maintained the ladder at the required depth. From the nature of the work, this ladder had to act vertically, involving thereby the use of buckets, with their backs hinged in such a manner that, when

passing over the cam, their backs were turned inwards by arms on the cam shaft, and thus each bucket was completely emptied, and the earth so excavated fell into a shoot which conducted it to a barge. The dredger remained fixed in the centre of the caisson, the superincumbent weight of which was sufficient to force more material into the hole already formed by the buckets. The ladder had a vertical travel adjustable to the depth required to be excavated. This apparatus could excavate about 10 cubic yards of material per diem, at a cost of 8*s.* per cubic yard, and was the most rapid and least costly of the methods employed.

Taking the average of several caissons lowered by the ordinary method of sinking, namely, where the soil is excavated and raised in skips, while the engines and pumps keep the excavation free from water during the interval of lowering, it appears that a caisson could be sunk to a depth of about 20 feet in eight days and one third, the total quantity of soil thus excavated during twenty-four hours being about 6 cubic yards. The average cost of labour by this method was 14*s.* 6*d.* per cubic yard. When the caissons were sunk by the pneumatic system, the average time occupied in sinking to a depth of 20 feet below the bed of the river was eleven days and a half, which gives about 4½ cubic yards as the daily average of work by this process, and the cost of labour per cubic yard 12*s.*

The average amount of work done, in a given time, by the ordinary and pneumatic systems of dredging were respectively 60 and 45 per cent. of the quantity executed by the telescope dredger. The cost of labour per cubic yard of soil raised in working the telescope and pneumatic machines, as compared with the cost of raising 1 cubic yard by the ordinary method, was only 55 and 83 per cent. respectively of the latter price; but the effect of the prime cost of these various mechanical contrivances on the foregoing figures has not been reckoned. One hundred and ninety-five of the elliptical caissons were sunk from the foreshore to the clay by dredging, and they traversed a distance of about 2,440 feet. Of this length 625 lineal feet of caisson were lowered by means of a bag and spoon, 1,337 feet by manual labour, without mechanical assistance, a gang of about ten men sinking each caisson; 212 feet by the dredging ladder; and 75 feet by removing the gravel to the clay by dredging and lowering the caissons into the trench thus excavated. The pneumatic apparatus was employed in sinking 187 lineal feet of caissons. The caissons, in all cases, after reaching the clay, were lowered by manual labour.

In comparing the cost of the iron caisson and ordinary clay and

timber cofferdams, the Author has thought it best to consider them as if employed under similar conditions. As a basis for calculation he has taken 30 feet 6 inches below ordnance datum as the mean depth for sinking both kinds of cofferdam. In order to prevent overflow, the tops of the cofferdams were fixed at 4 feet above Trinity high-water mark, so that the iron and timber cofferdams both measured 47 feet in height. The kentledge employed for weighting the caissons was supplied at about £4 15*s.* per ton. and was resold by the contractor, after the works were completed, for £1 16*s.* per ton. The average cost of labour in concreting the caissons was £3 15*s.* 9*d.* per caisson, or for a length of 12 feet 6 inches of dam. The cost of staging for the dam was about £3 per lineal foot. The quantity of timber consumed per lineal foot in the construction of the clay and timber cofferdam was 117 cubic feet, and in the iron cofferdam, 63½ cubic feet, the latter amount being 54 per cent. of the former.

In the iron cofferdam 39½ cubic feet of timber per lineal foot of dam were consumed in the walings, staging, and piles, and the remaining 24 cubic feet per foot of dam, in the 25-feet bays, over which worked a traveller, having a span of 55 feet. In calculating the price of the iron cofferdam, the cost of that portion of the iron not permanently used in the foundations, is taken as having been twice employed. The three upper courses of this iron, as already stated, were half rings only, so that, supposing the top level of the iron permanently buried to 14 feet below ordnance datum, the upper portion of the dam removed would form a second complete caisson 23 feet 9 inches in height. The total cost of the timber portion of this dam, including piling, shoring, &c., and 4*d.* per cubic foot depreciation of plant, amounted to 2*s.* 6*d.* per cubic foot.

The gross cost per lineal foot of the iron caisson cofferdam was :—

		£.	s.	d.	£.	s.	d.
Timber, 63½ cubic feet	at	0	2	6	7	18	9
Iron in caisson (nine rings = 1·08 ton) .	„	16	15	0	18	1	9
Excavation, allowing profit for gravel (5 cubic yards)	„	0	5	8½	1	8	6
Kentledge, used many times over, 1·6 cwt.	„	0	4	9	0	7	7
Labour of concreting per caisson	„	3	15	6½	0	6	0½
Portland cement in caisson, 10 bushels .	„	0	2	0	1	0	0
Wrought iron, 44 lbs..	„	0	0	2	0	7	4
Cost of dam per lineal foot					29	9	11½
Cost of removing dam					0	17	6
					30	7	5½

Deductions from gross estimate per lineal foot:—

		£.	s.	d.	£.	s.	d.
Value of iron used a second time in caissons, or 0·30 ton	at	16	15	0	5	0	6
Contractor allowed £8 per lineal foot in all cases for caissons left in as permanent work.					8	0	0
Kentledge resold, 1·6 cwt.	„	0	1	9	0	2	9½
Old caisson rings resold, 0·30 ton	„	1	16	0	0	10	9½
Timber removed, 63½ cubic feet	„	0	0	8	2	2	4
Total deductions from gross estimate. . .					15	16	5

	£.	s.	d.
Gross estimate	30	7	5½
Deducting	15	16	5
Total net cost of caisson cofferdam per lineal foot	14	11	0½

The gross cost of the timber cofferdam, as shown by Mr. Ridley, was:—

		£.	s.	d.	£.	s.	d.
Dredging					0	7	0
Stage for dredging and piling					1	4	0
Timber, 117 cubic feet	at	0	2	6	14	12	6
Cast iron, 143 lbs.	„	0	0	1	0	11	11
Wrought iron.	„	0	0	2	0	9	10
Puddle, 9 cubic yards	„	0	2	3	1	0	3
Clay backing to outer and inner faces, 5 cubic yards	„	0	1	2	0	5	10
Cost of dam per lineal foot.					18	11	4
Removal of dam.					1	4	0
					19	15	4
Deduct value of iron and timber removed .					2	10	6
Net cost of dam					17	4	10

Thus the net cost of the iron caisson dam was £2 13*s.* 9½*d.* per lineal foot cheaper than the ordinary wooden cofferdam. But as the iron caisson cofferdams were chiefly employed at Westminster, Charing Cross, and Waterloo steamboat piers, and at York Gate and the Adelphi landing stairs, where the works were of special difficulty, it would be more correct to compare the cost of this iron cofferdam with that of the timber and clay one used at the Temple pier. The cost of the latter was £20 15*s.* per lineal foot, or £6 3*s.* 11½*d.* more than that of the iron caisson cofferdam. Before the contractor was recouped on his original outlay by the sale of

plant, the total cost per lineal foot of the timber and iron cofferdam, as they stood upon the works, showed the timber cofferdams to be £10 18s. 7½d. per lineal foot cheaper than the iron one; but this apparent economy forms no correct criterion of the intrinsic cost of the cofferdams to the contractor.

It has previously been remarked that, after the caissons were reremoved, a large portion of the surplus iron was converted into pontoons for the construction of landing stages for the steamboat piers, which cost about £33,000. The profit reaped by the contractor by the employment of his surplus stock has not been taken into consideration in estimating the cost of the iron dams.

The aggregate length of the three contracts between Blackfriars and Westminster bridges is 6,641 feet; and the cost of the several works as tendered for by the three contractors, was—Mr. Furness, £520,000; Mr. Ritson, £229,000; and Mr. Webster, £126,000, making the total contract price £875,000.

Some idea of the magnitude of these works may be gathered from the following summary of the quantities of the materials employed in their execution, which were approximately as follow:—

Granite	650,000 cubic feet.
Brickwork	80,000 „ yards.
Concrete	140,000 „ „
Timber (for cofferdam)	500,000 „ feet.
Iron caissons (for do.)	2,000 tons.
Earth filling	1,000,000 cubic yards.
Excavation	144,000 „ „
York paving	125,000 square feet.
Broken granite	50,000 superficial yards.

The Author is indebted to Mr. George Furness, and some of his assistants, for many details of cost given in this Paper.

THE ALBERT EMBANKMENT (Plate 2, Figs. 11 and 12).

The Albert Embankment between Westminster and Vauxhall bridges, constructed under the Act obtained in 1863, was carried out under the immediate direction of Mr. John Grant, M. Inst. C.E., who has so largely contributed to the improvements in the manufacture of cements and concrete. It was commenced in September 1865, and opened to the public in May 1868. It extends along the river for a length of 2,200 feet between Westminster and Lambeth bridges, and for a further length of 2,100 feet from Lambeth Bridge to the site of the London Gas-works. It was originally

intended to be extended 1,000 feet further to Vauxhall Bridge, but this portion of the work was abandoned for lack of funds.

The wall is straight from Westminster Bridge to Lambeth Bridge, and the remainder curves to a radius of 21,120 feet, and with very few exceptions is of uniform character. It is similar in elevation to that on the Middlesex side, having a highly-dressed granite facing, with plain curved battered surface up to high-water mark, and above that a moulded parapet and plinth, the mouldings being stopped at frequent intervals against plain pedestals of granite, furnished with ornamental bronzed mooring rings, similar to those on the Middlesex embankment, and surmounted with standards for gas-lights. A small portion of this wall differs from that on the Middlesex side, in being formed with concrete faced with granite. A subway and part of the low level sewer was, however, constructed in the Victoria Embankment, so as to form a portion of the structure of the wall.

There are no recesses for pontoon steamboat piers in connection with the Albert Embankment, but the approaches to the Lambeth steamboat pier have been improved and enlarged. At Westminster Bridge, a landing-place for smaller boats has been constructed, and there is a wide flight of steps from the bridge to the 20-feet promenade footway formed along the embankment wall. At the pottery works of Messrs. Doulton and of Mr. Stiff, docks have been formed on the land side of the embankment roadway. The entrances pass beneath the road without interfering with its level. Openings, or draw docks have also been formed further up the river, to give access for carts and wagons, and to accommodate the traffic generally.

The foundations of the river wall are carried down to a level of 30 feet below Trinity high-water, and the foreshore has been excavated to a depth of about 18 feet below the same datum. The reclaimed land afforded a site for the construction of St. Thomas's Hospital. The bulk of this work has been built behind a whole-tide timber cofferdam of the ordinary type; but at about 800 feet above Lambeth Bridge the wall was constructed in a trench excavated out of the solid ground, for this part of the wall ran inland. Here the ground on the river side has been removed, the space being thrown into the river, so as to increase its width from 600 feet to 720 feet. At Westminster Bridge the width is about 800 feet.

A length of 1,050 feet of this embankment above Lambeth Bridge was constructed behind a dam formed with a single row of piles closely driven and caulked. The borings showed the clay

to be nearer the surface along this length than at Westminster Bridge, and a single-pile dam was thereby rendered practicable. The promenade in front of the embankment is 20 feet wide; and the roadway behind it, from Gun House Alley to Westminster Bridge, is 60 feet in width. Gravel was reached at a moderate depth, so that the foundations of this embankment were more easily obtained than on the northern side. The structural cost of the work was £309,000.

THE CHELSEA EMBANKMENT (Plate 2, Figs. 13 and 14).

It was proposed in 1839, by the Commissioners of Her Majesty's Woods and Forests, to form an embankment on the Middlesex shore of the river between Vauxhall and Battersea Bridges; and in 1845 the Commission for the Improvement of the Metropolis reported in favour of this scheme, and recommended its execution at a sum of £92,000. Of this amount, £43,000 were to be contributed by the owners of river frontage property, and the remainder was to be provided out of the public purse. After a delay of seven years, an Act was obtained authorising the issue of Exchequer Bills for an amount not exceeding £120,000, to enable the Commissioners of Her Majesty's Woods to construct the embankment and roadway, and to build a suspension bridge across the River Thames opposite Chelsea Hospital. A sum of £146,000 was raised, and the bridge built; but the cost was more than had been anticipated. The balance, with the exception of £38,150 returned to the Treasury in 1857, was only sufficient to form the embankment and roadway as far as the western end of the Chelsea Hospital gardens, and the remainder of the work was abandoned.

No convenient thoroughfare, however, existed between Cremorne and Chelsea Hospital for this portion of the Main Drainage Low Level Intercepting Sewer, of which the construction was originally contemplated beneath the foreshore of the river; but as this plan involved its formation behind a temporary dam—in itself a costly expedient—the preferable method of forming the sewer behind a solid embankment was proposed, and eventually carried out. The Metropolitan Board of Works applied to Parliament in 1865 for powers to execute this work; but, the money bill not having been introduced by Government that session, the Bill was withdrawn. A similar application, which resulted in failure, was made in the course of the ensuing session, and the Act was not obtained until 1868.

This work was commenced in July, 1871, and was finished in May, 1874. It extends from Chelsea Hospital to Battersea Bridge on the northern bank of the river, and is upwards of ¾ mile in length. With the exception of a short length of river not embanked, between Millbank and 360 feet west of the Houses of Parliament, it completes one continuous river embankment extending from Battersea to Blackfriars Bridge, a distance of 4¼ miles. The works comprised the construction of 4,130 lineal feet of river-wall formed with granite facing backed with Portland cement concrete in the proportions of 6 parts of ballast to 1 part of cement from the foundation level up to ordnance datum, and above that level of 8 of ballast to 1 of Portland cement; also the construction of 4,430 lineal feet of low level brick sewer, varying from 5 feet 9 inches to 6 feet 9 inches in diameter, which connects the drainage from Fulham and Hammersmith with the Abbey Mills pumping station at the extreme east of London. The line of wall is straight from the Chelsea Hospital to above the Albert Bridge, and the remainder is to a radius of 5,100 feet. The width of river previously existing along its length has been reduced from between 700 and 850 feet to a nearly uniform width of 700 feet.

An area of 9½ acres of foreshore, formerly covered with an average depth of 4 feet of mud, has been reclaimed by the embankment. This space is now occupied by a roadway 70 feet in width, the remainder being laid out as ornamental gardens. The roadway is 5 feet above Trinity high-water, and the foundations of the wall are carried down to an average depth of only 4 feet below the level of low-water spring tides, or 10 feet below ordnance datum. They were got in behind a half-tide dam.

A trench was first excavated to the required depth, the bottom of which was levelled and prepared in short lengths to receive the concrete. Blocks were bedded upon the ground with fine cement concrete as the tides permitted. They were prepared in boxes at least three months previous to their employment in the works, and were composed of 6 parts of ballast to 1 part of Portland cement. The concrete was filled in at the back of the wall, so that as the granite facing bonding one header to every three stretchers was brought up, it was securely keyed into and incorporated with the concrete filling. As soon as the tide ebbed level with the tops of the piles forming the dam, the enclosed water could be rapidly got rid of by pumping. As the work was executed in different stages, when the masons were compelled by the flood tide to relinquish operations on the lower levels of the works, they could be concentrated upon the more advanced portions less subject to tidal

influence. The dam was provided with sluices in the ordinary manner, and the tide, after flowing to a given height, was always admitted within it.

The granite facing, instead of being dressed to a smooth surface as in the other embankments, was hammer-dressed, and the parapet was made of a bolder and less refined contour. It is dressed on the river side to correspond with the general appearance of the wall. The result has been effective at a reduced expense. The total cost of the structural work, including the low level sewer, was £134,000, or about the cost of a whole-tide cofferdam for the Victoria Embankment. The introduction of concrete, in lieu of brickwork, effected a saving in this embankment of £21,000. Since its formation a settlement occurred in a short length opposite Cadogan steamboat pier, caused, it is supposed, by the removal of some piles which formed a part of the old pier in front of it. The wall has since been underpinned from the land side, and on the other side the toe has been protected by sheet-piling.

The more minute details of the construction of the Chelsea and Albert Embankments could not be treated within the limits of one Paper; but the work was generally similar in nature to that of the Victoria Embankment.

The Paper is illustrated by a series of diagrams, from which Plates 1 and 2 have been compiled.

[Mr. E. Bazalgette

Mr. E. Bazalgette desired to add a few remarks with regard to the formation of the caissons, the details of which had not been described. They were formed of wrought-iron plates, about ½ inch or ⅜ inch in thickness, lap-jointed and bolted together. The Victoria Embankment wall had cost £39 per lineal foot, exclusive of the wooden cofferdam. In comparing the cost of the caisson and of the timber cofferdams a certain standard of height had been taken—they were both supposed to extend from a level of 30 feet 6 inches below ordnance datum up to a level of 16 feet 6 inches above datum, making a total height of 47 feet. As the caisson and timber cofferdams were erected close to the foundations of the intended river wall, and extended considerably below them, the piles, when the dams had done their duty, had to be cut at levels between 3 and 7 feet under low water—3 feet along the general line of the wall, and 7 feet opposite the steamboat piers, where a greater depth was required. Under those circumstances the piles could not be used a second time. The advantage of the iron cofferdams was that the portion not permanently buried in the work could be used several times over. In the Paper they were considered to have been used only twice. A sum of £8 per lineal foot was allowed the contractor for that portion of the iron cofferdam permanently buried in the work, but no such allowance was made for the corresponding portion of the timber cofferdam. If, therefore, in arriving at the net cost of the two dams, it should be considered more correct to disregard that allowance in respect of both dams, the net cost of the caisson cofferdam, instead of being £14 11*s*. 0½*d*. per lineal foot, would be £32 14*s*. 0½*d*., or £1 16*s*. more than the timber dam erected opposite the steamboat pier which had cost £22 15*s*. per lineal foot. It was believed, however, that the contractor effected considerable economy by the employment of iron cofferdams instead of timber, for not only was the upper portion of the caisson used in many cases three or four times over, but after the removal of the dams a large portion of the surplus iron was utilised in constructing pontoons for the different steamboat piers, which was a profitable way of employing surplus stock. Of the three embankment works, No. 1 contract of the Victoria Embankment was the only one where iron dams were used. That contract extended from Westminster Bridge to 300 feet east of Waterloo Bridge, and comprised a length of river wall of 3,800 lineal feet, of which about 2,400 feet had been put in by the caisson dam. The double covered way, described in the Paper, which passed through the Victoria Embankment close to Blackfriars Bridge, for the purpose of facilitating the traffic of

the City Gasworks, was no longer in use, as the gasworks had been removed.

Mr. ABERNETHY, Vice-President, said the Paper purported to be upon the Embankments of the Thames, hence one might infer it related to the general question of embankments. It was partly historical, but was mainly confined to a narrative of the various works carried out on the Thames without reference generally to the effect of the embankments on the river. It was an acknowledged fact that by the removal of Old London Bridge and of other obstructions, and by dredging the various shoals, the hydraulic mean depth of the river had been increased; consequently the velocity of the tidal current flowing in and its volume had likewise been increased, the effect being to raise the high-water level and to depress the low-water level. To that extent the river Thames had by those operations been improved, but undoubtedly much remained to be done. The great variation in the sectional area of the river, arising from the constant variation in its width, yet remained to be rectified, in order to provide a regular continuous channel. There was still a certain amount of obstruction at all the bridges, a heaping up of the water surface in flood tides to a greater or less extent. The Author had remarked that "although mud banks of considerable extent exist upon the Surrey side, between Westminster and Blackfriars, and the embankment of that shore would much improve the appearance of the river, yet the advantages to be obtained by such a work would not be equal to those on the north side; for no public roadway is needed there, nor any sewer, and the claims for compensation to the owners of the wharves would be heavy" (*ante*, p. 3). In his opinion it would be desirable that that embankment should be carried out. The question of compensation to the wharf-owners could easily be arranged, by allowing them to partake of the benefits arising from the reclamation of the foreshore. With regard to the mud banks, he would not touch upon that question or he might be overwhelmed by learned counsel as on a former occasion. The Author had referred to the floods of the river Thames. It had been and was still asserted that the formation of the Thames Embankment had tended to raise the level of the surface of the water in periods of floods, and to cause an overflow of the banks that did not formerly occur. He was one of the engineers who did not entertain that opinion. In the first place, floods had occurred in the river from time to time for centuries. Long after the Embankment had been formed, in June 1875, floods occurred in the lower reaches of the river at Millwall Docks, more than 3 miles [illegible] the lower end of the Embankment, and at the Victoria Dock

8 miles below, with which the Thames Embankment, at any rate, could not have had anything to do. Again, the sectional area of the water-way, say at Southwark Bridge, was less than the sectional area at the Thames Embankment in the proportion, he believed, of 14 to 19. The embankments of the Thames had increased the hydraulic mean depth at that point, consequently the velocity of the current, and the volume; but the diminution of the sectional area and the increased velocity were only equivalent to the greater sectional area and the lesser velocity of the river below. That, he thought, showed that the Thames Embankment was not the cause of the flooding. The increase in the velocity of the current caused by the embankment tended, at periods of floods, to accelerate the discharge; and the upland waters, which had increased by the great extension of the drainage of the Thames valley generally, were also relieved more quickly than formerly, when the river was of a less hydraulic mean depth, and when the current was deflected and impeded by numerous barges lying upon the river banks.

With regard to the caisson dams, a cofferdam being a mere temporary work, a mere implement, he was of opinion that the greatest economy should be studied in constructing it, and more particularly in regard to time. The Author had stated, "Taking the average of several caissons lowered by the ordinary method of sinking, namely, where the soil is excavated and raised in skips, while the engines and pumps keep the excavation free from water during the interval of lowering, it appears that a caisson could be sunk to a depth of about 20 feet in eight days and one-third" (*ante*, p. 17). Each caisson would probably be equivalent in length to ten or twelve piles, and two rows of such piles could be driven into soil of that character in a much shorter time than eight days. In all such matters he was an advocate for simplicity of construction. The Author had given a comparative statement of the cost of the two processes, by iron caissons and by the usual timber piling, showing the total net cost of the caisson cofferdam per lineal foot to be about £14 11*s.* 0½*d.*, and the cost of the wooden cofferdam to be £17 4*s.* 10*d.* The value of the iron and timber removed in the latter was estimated at £2 10*s.* 6*d.*, whereas the value of the iron and timber originally used in the cofferdam was £15 14*s.* 3*d.* The timber must have been much destroyed, and the iron also, if their value were reduced to £2 10*s.* 6*d.* He thought that figure was an under-estimate, and he believed if the item were fairly given the timber cofferdam would be found to be the cheaper of

the two. At any rate, it was evident that in the important element of time the timber dam was to be preferred.

With regard to the Chelsea Embankment, the Author had stated that the removal of some piles forming a part of the old pier had been the cause of a portion of the wall giving way. In another part of the Paper it was stated that the foundation of the wall was only 4 feet below the level of low water, therefore, considerably above the level generally of the bed of the river. He apprehended that the failure arose from the fact that there was not sufficient footing to resist the lateral pressure of the embankment behind. That was an error into which many other engineers had inadvertently fallen.

Mr. Shelford wished to thank the Author for giving the members another opportunity of discussing the question of the effect of the Thames Embankment upon the high-water line of the river. With regard to the details of construction, the Author had stated that the cost of the iron caisson cofferdam was less than that of the timber cofferdam. He had been under the impression that iron caisson cofferdams had been abandoned on account of their cost, and he hoped the Author would state the reason for their use during only the first portion of the work and their subsequent discontinuance. It had also been stated that, of the three modes of excavating material from the inside of the iron caissons, that was found the cheapest and best which excavated by machinery under water, the water being allowed to rise and fall in the cylinder during the operation. He believed that experience had since been borne out in the construction of the bridge over the Clyde, where cylinders were sunk, the water rising and falling in them, by means of Milroy's excavator. More recently at the river Tay bridge, just completed, after several trials had been made, the mode ultimately adopted was the sinking of large wrought-iron cylinders 31 feet in diameter as the foundation for each pier, and excavating the materials from within by means of a flexible suction pipe discharging into receivers from which the air had been exhausted. It had been patented by Mr. F. W. Reeves, Assoc. Inst. C.E., and was worked under water, the tide rising and falling within the cylinders. The Author appeared to admit that the Embankment had not been constructed so high as it would have been had the present height of the tide been anticipated. Sir Joseph Bazalgette and Mr. Walker, whose lines appeared to have been adopted, were not singular in that respect, for it had frequently happened that engineers had ad-

mitted more tidal water than they anticipated; and he thought it might be accounted for in this way. The effect of the removal of the bridges over the Thames, and, in a less degree, of the construction of the embankments, would undoubtedly be to raise the water line to the same level above bridge as below bridge; and from a former Paper by Mr. Redman[1] he gathered that such was actually the case. But besides that there were, in the complicated state of things arising in the upper part of a tidal river, other causes which might possibly affect high water; and the cause he would particularly refer to was the wave action of the tide—the great tidal wave, of which high-water mark was the crest, which advanced more rapidly up a stream in deep water than in shallow water. It was long since Professor Airy published an elaborate theory of the tides, which had been well worked out and reduced to a table. It appeared that in the open sea the velocity of waves of the same breadth increased in the ratio of the square root of the depth. But that was not always the case in rivers. On the Humber he had found that with an increase of depth, arising from the difference between neaps and springs, of one-sixth, there was an increase in the velocity of the crest of the tidal wave (or high water) of one-third. Was it not possible that engineers had sometimes misunderstood the theory which had been so ably propounded by Professor Airy, or had not been able to obtain actual data, or had disregarded them if obtained, and had not attached sufficient importance to the increased wave action arising from the increased depth? That increased action, he submitted, might be insignificant in a river where there was no interruption to the tide by a dam or weir, or where there was no fresh water flood, but it did lead to a super-elevation of the high water when the tide was interrupted by a weir or dam, or where the tidal wave met a fresh water flood. He had referred to the Humber because on one of its principal tributaries, the Ouse, there existed such a weir (Naburn Lock), and through the other, the Trent, much fresh water was discharged. Observations had been taken both at Naburn and Gainsborough by a Committee of the British Association, a summary of which, by Mr. Parkes, was appended to his Paper on the "Outfall of the Humber."[2] At Naburn the range of tide was now 8 feet as against 5 feet before the erection of the lock; and at Gainsborough a 25-feet tide, at Hull, rose 4 inches above that of Hull, when there

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xlix., p. 67.
[2] *Ibid.* vol. xxviii., p. 512.

were only 8 inches of fresh in the Trent; but it rose 21 inches above that at Hull, when there were 36 inches of fresh; and the velocity of the tidal wave was then rather increased than diminished. Although he had no doubt that the high water line in the upper part of the Thames was raised by the increased momentum (due to the removal of the bridges, &c., and in a less degree to the construction of the embankments) of the tidal wave against Teddington Lock, he did not suppose that this influence would be felt far down the river. But the super-elevation above a level line of high water, caused by a spring tide rising against a fresh water flood, would be much greater at Teddington than at Gainsborough; and might extend as low as the metropolis, and prove to be the cause of the floods which had attracted so much attention.

Sir JOSEPH BAZALGETTE, C.B., said the three embankments under consideration extended for a distance of 3½ miles along the river, and they had reclaimed about 52 acres of land. Their object and their mode of construction varied considerably. It was not his intention to enter fully into what might be considered the effect of those embankments upon the *régime* of the river Thames, or upon the rising of the tides in recent years, that subject having been discussed at the Institution a few months ago, when it was stated that the embankments were the cause of the high tides and the flooding of the lower districts.[1] He believed that view was supported by a non-professional gentleman, a member of Parliament, but it obtained no favour among the members of the Institution. The subject had been fully discussed before a committee of the House of Commons, and the result of the investigation was, he believed, such as confirmed the view expressed by Mr. Abernethy, that the embankments had not been in any respect the cause of the increased high water in the river. That there had been an increase of tide of late years no one could dispute. Mr. Abernethy had properly attributed it to the effect of the removal of the old bridges, of the obstructions by dredging, and of the admission of a larger body of tidal water into the river. Thirty or forty years ago, Mr. James Walker, who was consulted by the Trinity Board, laid down a line for an embankment between Westminster Bridge and Blackfriars Bridge, and determined the safe height to which such embankment should be constructed, viz. 4 feet above Trinity high-water mark. That was the height which he believed most engineers up to the present time had adopted. He himself had adopted it, without further investigation, as the

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xlix., p. 67.

safe line for the Victoria and Albert Embankments; but subsequent experience and examination had induced him to adopt 5 feet instead of 4 feet for the Chelsea Embankment, and to recommend that in future all wharves should be raised to that height. The Victoria Embankment had engaged the attention of most members of the profession during the last half century; and he supposed there was no work for which a larger number of proposals had been made. There had been proposals for high-level roads, with docks and passages underneath, and for docks without roads in front of them; there had been designs for utilising the land by constructing buildings upon it; and almost all those designs had been governed by the necessity of making the land reclaimed (about 37 acres) remunerative. Now that object would have rendered the Embankment certainly unsightly and less advantageous to the metropolis; and he thought it a matter for congratulation that none of those designs had been carried out, but that it had been left to a public body, having funds, to expend those funds for the public good. The modes of constructing the embankment had also been various. The feature of getting in the foundations had been alluded to by previous speakers. He might, however, mention that between Westminster Bridge and Waterloo Bridge the foreshore of the river presented much greater difficulty in getting in the foundations than any other portion—greater difficulty than had been experienced in forming the foundations of the Albert or the Chelsea embankments. It was the knowledge of that difficulty which had induced him to recommend iron caissons in lieu of piling for that portion, because those caissons formed a part of the permanent work. They had been spoken of as mere temporary dams, but they were neither constructed nor used with that object alone. They had been carried down into the solid clay, and they enabled a foundation to be obtained at a less depth than that at which it would have been otherwise obtainable. There had also been a great advantage in constructing the work with caissons near Westminster and Waterloo bridges, where the driving of piles would probably have shaken those structures and have been dangerous. The members were no doubt familiar with the various modes in which caissons had been sunk. They could be sunk without any vibration or damage to the surrounding buildings. Three modes of sinking had been tried, all of which were of considerable interest. One was by men entering the caissons and pumping the water out of them. The next was by the pneumatic process, the caisson being carried above high-water level, and a chamber being fixed at the top into which

a man could enter; then, having closed a door above him, he would open a door in the caisson below into which the air was compressed; it would then ascend into the chamber in which he was placed, giving a uniform pressure; then he carried on his work, and having brought up the gravel or sand from the bottom into the chamber, he would close the door again, and then, by the escape of only so much pressed air as was contained in that small chamber, the earth was taken out. The third mode was by a telescopic dredger. That was found to be the cheapest method of effecting the dredging. A chain with scoops passed round a beam, which beam was from time to time lengthened as it was found that the scoops were not doing their work. Thus the dredging was carried on without pumping out the water. The cost by the first mode was 14*s.* 6*d.* per cubic yard, by the second, 12*s.*, and by the third, 8*s.* He believed his statement on that subject accorded with the experience of others who had made the experiment. By the use of the method he had recommended, it was possible to place new foundations below old piers of bridges, or to place foundations in the neighbourhood of buildings where pumping could not be resorted to with safety. That plan was now being more generally used, and he believed it would be still more employed in future ages. The question had been asked why the caissons were used only in that part of the work between Westminster and Waterloo bridges. There were one or two reasons. In the first place, in other portions of the work, the solid ground was reached more easily, and, to make it pay, it was necessary that there should be a sufficient number of caissons to use them at least twice over; and if the work had to be carried on rapidly there was a difficulty in that respect; some additional work was therefore required, such as foundations, to induce an engineer to use caissons. If they had been used between Waterloo Bridge and Blackfriars Bridge the work would have been delayed. In the Albert Embankment the clay was reached at a less depth, and it was found that a single whole-tide dam, well caulked, was sufficient, and that of course was much cheaper than a double dam with clay between the piles. In the Chelsea Embankment the work was carried out with merely a half-tide dam. In the Victoria Embankment the depth of the foundation was 14 feet below low-water mark, in the Albert Embankment 11 feet, and in the Chelsea Embankment 4 feet. In the Victoria Embankment the foundations did not represent the whole depth, because in bad places they were protected by the caissons. In the Albert Embankment the river had been widened considerably. Instead of being em-

banked in the ordinary sense the shore was excavated, and a larger channel thrown into the river, and that being the narrowest point considerable advantage was gained. With reference to the Chelsea Embankment, the foundations, as he had said, were only 4 feet below low-water mark. It had been suggested that it was the want of depth which had caused a portion of that embankment to settle and give way. The whole length of the Chelsea Embankment was 4,300 feet, and all the foundations were laid at a uniform level, the gravel being of the same character throughout. The embankment was opened to the public in the spring of 1874. The foundations had been got in and had been weighted about eighteen months before. In the autumn of 1875, eighteen months after the opening, and three years after the completion of the foundations, the embankment stood firm without any appearance of settlement; but at that period thirty piles, which were close in front of the portion of the embankment which gave way, and were driven 6 or 7 feet deeper than its foundations, were drawn. These piles had formed part of the old Cadogan pier; they were close to the embankment, and were drawn by the Thames conservators from the front of the embankment wall. In addition to the thirty piles a portion of the brick and iron work of the old foundations was removed. In the spring of 1876, six months after the piles had been drawn, settlements were observed in the embankment opposite where the piles had been, and it was found that the settlement extended 300 feet along the embankment, corresponding with the line of the piles. The remaining 4,000 feet of the embankment continued firm, as it did to the present day. The 300-feet length of embankment was underpinned and made good, and it also was now perfectly complete and uniform. He admitted that the evidence was circumstantial; but he thought it was sufficient to enable anyone to determine the causes which had brought down that portion of the embankment wall.

Mr. Redman thought the embankments that had preceded those described in the Paper ought not to be ignored; for example, the Penitentiary embankment wall, by Sir Robert Smirke, 1,500 feet in length; the Houses of Parliament embankment wall,[1] by Mr. Walker, 1,200 feet; the Iron Gate and Fresh wharf, 800 feet; the Custom House wharf, by Mr. Laing, 600 feet; the Tower wharf, recently raised, 1,200 feet; the St. Catherine's

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. i. (1840), p. 18.

wharf, 500 feet; Deptford Dockyard wharf, 2,500 feet; Greenwich pier, one of the loftiest embankments of the river, in the deepest water, by Martyr, 500 feet; Greenwich Hospital embankment, 1,000 feet; Mowlem's wharf at East Greenwich, constructed by himself, 700 feet; Brunswick wharf, Blackwall, by Mr. Walker, 1,000 feet; Woolwich Dockyard, 3,300 feet; making a total length of 14,800 feet, or nearly 3 miles. In addition there were private quays from Blackfriars Bridge to the West India docks on the north, and private quays on the south from the same point down to the Commercial docks, representing an aggregate of 37,900 feet, or nearly 7¼ miles. So that in all there were about 10 miles of comparatively modern quays preceding the embankments in question. But those works were altogether thrown into the shade by the early embankments of the lower reaches of the Thames—what were locally called sea-walls—which might be considered pre-historic, there being no record of their formation. They were amongst the most marvellous works of their times, and had certainly been constructed centuries before Acts of Parliament were heard of. They were usually attributed to the Romans, and there were indications within the banks themselves of their great antiquity. Those embankments, about 100 miles in length on the two sides, excluded about 30 square miles of tidal water, from land that was from 4 to 7 feet below the level of high water, Trinity standard. The condition of the river before the formation of those embankments must have been altogether different from its present form. It had been assumed that the very name of London was derived from "Llyn-Din,"[1] or a town on a lake, and the probability was that before the formation of those embankments the high water at London did not exceed the level at sea. At present there was a difference of 5 feet, with a fair spring tide, between Sheerness and London Bridge, over a length of 48 miles, yielding a gradient of about 1¼ inch per mile. No doubt that had resulted from the early embankments. They excluded an enormous amount of tidal water, but they recouped the river to a certain extent in raising the high-water level, and in deepening the low water *régime* of the river. The effects produced by those embankments had been continued upward through the metropolis by the modern embankments, and the same gradient was continued to Teddington. The distance between London Bridge and Teddington lock was 19 miles, and the high-water

[1] *Vide* Minutes of Proceedings, Inst. C.E., vol. xv., p. 196.

level was 2 feet higher at Teddington, yielding the same gradient of $1\frac{1}{4}$ inch per mile.

He had recently read a Paper at the Society of Arts,[1] supplementary to that which he had read last Session before the Institution,[2] and having there fully advanced his views respecting the tides he would not dwell long upon that point. The exceptional tides that had recently occurred in the Thames were certainly remarkable, and as he had reflected a great deal on the question he would shortly state what he thought to be the reason for those tides, and for their accentuation during the last few years. Down to 1869 there had been no tide in the Thames exceeding 3 feet 7 inches above Trinity standard, and that period was nearly coincident with the completion of the Victoria Embankment. For five years subsequently there was no tide of any great height; but in March 1874 the height suddenly increased 9 inches—to 4 feet 4 inches above Trinity. In November 1875 there was another tide 4 feet 9 inches above Trinity standard, or 14 inches higher than any preceding tides, excepting that of March 1874. Those tides had resulted from the concurrence of three causes which were very exceptional, but which must have occurred in all time; (1) an unusual heavy land flood meeting (2) an equinoctial spring tide, accompanied by (3) a strong westerly wind acting upon the tidal wave of the English Channel, veering suddenly to the north, and performing the same office on the tidal wave in the North Sea—the two raising abnormally the tide of the Thames. It appeared to him that there were four reasons for the modern accentuation of abnormal metropolitan tides. In the first place, the land floods were the same. The amount of water was no greater than it was years ago, but it came down more quickly, and there was a larger amount to be delivered in a given time, owing to the extension of modern arterial drainage and sub-soil drainage. Secondly, the removal of Old London Bridge, and subsequently of Blackfriars and Westminster bridges, admitted a vast volume of additional tidal water, adding, in fact, 33 per cent. to the quantity of water flowing above London Bridge, high water now being 12 inches higher above London Bridge than it was in 1830, and low water being depressed 3 feet 6 inches. That second cause had resulted in a third—the additional volume had deepened the low-water *régime* of the lower metropolitan

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[1] *Vide* Journal of the Society of Arts, vol. xxvi., p. 289.
[2] *Vide* Minutes of Proceedings, Inst. C.E., vol. xlix., p. 67.

reaches, and that had been aided by dredging. The head of the great body of tidal water, with a minimum of 20 feet at low water, that came up the river from the sea twenty-five years ago was opposite the Arsenal at Woolwich; now it was 2 miles higher, above the Dockyard, due no doubt to the removal of the lower shoals in the river. The result was that a big spring tide coming up the river acted upon a deeper cushion of water, and the high water was consequently raised. Then there was a fourth cause—the modern Thames embankments. The Author of the Paper had stated that it was a popular opinion that the embankments had raised the water. He certainly endorsed that popular notion, but not to the popular extent. He had shown that there were four causes at work, and the question was to assign the proper quantity of water to each of those causes—a complicated process, and one as to which he did not possess the elements of calculation. The Author had referred to certain evidence given before a Committee of the House of Commons as settling the question, but he had omitted to state that that evidence was entirely *ex parte*. He had read carefully every word of it, and it had certainly led him to draw a somewhat different conclusion. The modern Victoria and Albert embankments had shut out, in round numbers, 700,000 tons of tidal water. He estimated that they had raised the surface of the water about 4 inches, and that amount of raising, taking the length up to Teddington Weir, and an average width of 500 feet, gave nearly an equivalent amount of water, about 700,000 or 800,000 tons. The embankments were in fact doing a great deal of good, they were recuperative, and they afforded in themselves a compensation far better than the tidal compensation reservoirs suggested by the Embankment Commission of 1861. With reference to the height of the embankment, 4 feet, it had been frankly acknowledged by Sir Joseph Bazalgette that he had taken Mr. Walker's height without much consideration, and that he now recommended 5 feet. Another eminent member of the profession, Mr. Bidder, Past President Inst. C.E., in laying out the Victoria Docks, had taken the same height; but that height, adopted in 1840, ought not, after twenty-five years' experience, to have been hastily chosen, there being many hints to show that the condition of the river had varied materially. The late Lord Palmerston, who showed the same sagacity in reference to the Thames Embankment as he had done in diplomacy, gave his adhesion to any embankment measure conditionally upon both sides of the river being included. There was practical evidence that the authorities had been taken aback in regard to the

present condition of things. The general height of the embankment was 4 feet above Trinity standard, with a parapet of 4 feet in addition, so that if it were continuous the total height would be 8 feet; but the continuity of the parapet was interfered with by the approaches to the steamboat landings. After March 1874, granite sills, 6 or 7 inches in height, were placed round the top of the landings of the steamboat piers. After the tide of November 1875 they were duplicated in width and in height. In addition there were double inclined planes landward of the axes of the oscillating stages rising to the same height of 14 inches. Then, thirdly, small cubes of granite had been inserted in the interstices between the small bases of the balusters of the open balustrade to keep the water out. Practically therefore the embankment was about 5 feet 2 inches above Trinity standard. He questioned whether that was a minimum height above which no tide would in future flow. From what he saw of the action of the present embankment he thought when the Surrey embankment was completed (which it must be sooner or later) that minimum would in all probability be 5 feet 6 inches. Two arguments had been used, which appeared to him to be fallacious, to show that the embankment had had no influence on the tides. One was that Southwark Bridge gauged the quantity of water passing up as it did before the embankments were constructed. It was true that Southwark Bridge gauged the quantity of water passing up the river as it did fifty years ago; but the quantity that passed up twenty-five years ago spread over a large area, the river being 1,300 or 1,400 feet in width, whereas the volume was now compressed into a narrower and deeper channel, and the surface was correspondingly slightly raised. There was a difference of 3 or 4 inches in the Mersey, caused by the embankment there; and there was a much larger difference on the Clyde, produced by similar works. The tidal observations, made from year to year by the late Mr. James Simpson, at Chelsea, showed that down to 1841 there was an absolute drop in the surface of the water at high tide. High water at Chelsea was 3 inches lower than high water at London Bridge, and the high water at Battersea was 6 inches lower. These conclusions were endorsed by tidal observations taken by the Metropolitan Sewers Commissioners in 1849, and they showed precisely the same fact—that high water at Chelsea at that period was 6 inches lower than high water at London Bridge. The other argument that had been urged was that there was a parallel increase at the Victoria docks and at other docks. That was not precisely the case. The tide of March 1874 was 4 feet above

Trinity at Blackwall, and 4 feet 6 inches above Trinity at Millbank. The tide of November 1875 was 4 feet 6 inches above Trinity at Blackwall, and 4 feet 9 inches at Westminster, showing the same progressive exceptional tide due to the causes he had imperfectly described. It appeared to him that these works were producing the results to which he had referred. As the embankments were continued up the river, no doubt the tide became greatly developed up to Teddington, because the rise of the high water was a gradient of 1¼ inch per mile, and low water was a rising gradient of 12 inches per mile. Low water at Teddington was 16 to 18 feet above low water at London Bridge, so that there was room for development by scour and by dredging. Ultimately, on the removal of Teddington Lock, which he believed would take place, and the carrying of the embankments higher and higher up the river, the tide might be continued to Moulsey, and the tidal properties of the river be immensely developed, not only promoting a sanitary advancement in London, but adding to its commercial importance. He thought the members were much indebted to the Author for this description of a great work that was producing very beneficial effects.

Mr. ALFRED GILES considered that, whether as citizens or as engineers, they all ought to be proud of the work which had been so ably described. Of the embankments in the lower reaches of the river but little was known. He had been astonished to hear there was a belief that the Victoria Embankment could have caused the tide to rise higher at that point than it did before. His own belief was that the cause of the rise of the tide through the metropolitan reaches was due to the removal of the obstructions of the bridges and to the great amount of dredging that had been done. If it were not so, he would ask how it was that there was no alteration in the level of high water below London Bridge, which was now much the same as formerly. It appeared to him that since the removal of Old London Bridge the tide had simply flowed further up the river and found its level; and if the pounding of London Bridge had had no effect upon the level of high water below the bridge, surely no appreciable effect could be produced by the Victoria Embankment. Everyone who knew the Thames was aware that the river was much narrower at Southwark Bridge, and at the Penitentiary, than opposite the Victoria Embankment, and that therefore there could be no pounding at the latter point. The remarks which had been made went to prove, that the embankment could have no effect upon the height of the tide, and it had been stated that the gradient of the stream up

to Teddington was the same as that below London Bridge. If, then, there had been any pounding or heaping up of the water by the Victoria Embankment, surely the gradient would not have been the same all the way to Teddington. He wished to ask whether the walls of the embankment could have been got in cheaper by independent cofferdams. The rise of the tide and the consequent strain upon the cofferdam was not so great but that a self-supporting dam might have been made at a cost of from £25 to £30 per foot. Sir Joseph Bazalgette had stated that the iron dam and the wooden dam formed part of the strength at the toe of the wall by being left in; but he maintained that for a wharf wall the leaving in of piles, although cut off to the bottom, was inadvisable. The concrete that had been mentioned was said to be in the proportion of 1 of cement to 6 of ballast. He had used a great deal of concrete, but he had never had the courage to use it so strong as that. He thought good reasons had been given why the cost of the three works should have varied so much. With regard to the marking of the tides, it was no doubt the fact that exceptional tides, arising from exceptional circumstances, would always occur. Some years ago he had marked tides at Southampton during a period of nine or ten months, and he thought he had obtained a good average; but his calculations were entirely upset by one storm, in which the tide rose 16 feet, being 3 feet 6 inches above the usual height. His experience was that averages calculated in that way would always be more or less incorrect, by the constant occurrence of exceptional circumstances.

Mr. Wentworth-Sheilds thought it would be difficult to exaggerate the importance and interest to engineers of the questions involved in the rise of the tides in the Thames; and he desired to make a few remarks on what he believed to be the general causes of such rise. In looking at what had been done in the Thames it appeared to him that the main features were as follows. First, the Thames Conservancy had executed an enormous amount of dredging in the shoal and irregular portions of the channel, and thereby produced precisely the same effect as would be produced in a water-pipe incrusted on the inside when the incrustations were cleared out, and an increased supply of water obtained. In that simple way, by the removal of the shoals, by straightening and regulating the course of the river, and by making the two banks parallel, as in the Victoria Embankment, a vastly increased body of water came up the river on the flood, and of course descended upon the ebb, to the great advantage of the navigation, not only in depth of water, but in the velocity of the stream,

which materially improved the condition of the important traffic by barges. These were the conditions on ordinary occasions; but on the other hand, on extraordinary occasions, when flood-water and spring-tides occurred together, an immense body of upland water came down the river, augmented by the improvement of the drainage of the watershed, and meeting as it did a high spring-tide coming up with a favourable wind, the effect was that of two hydraulic rams of several miles in length coming with great velocity and meeting each other at London, raising the fresh-water level by their impact. As the dredging continued so he believed the height of the tides would increase inch by inch, and a careful study of the effect of past operations in this way might afford a fair index of what was to be expected from future ones. He thought the historical part of the Paper was somewhat defective. It was known to many members that in the year 1861 the plan that had been executed was formulated and brought out by the labours of a distinguished Royal Commission, and that the subject was brought before Parliament by Her Majesty's Office of Works. He might be excused for saying that he had had some little share in the proceedings of that time, as would appear from the following extracts from the Report of the Royal Commission, viz.:—"The nature of the enquiry entrusted to us was made known to the public by advertisement in the newspapers, and more than fifty designs were presented for our consideration. . . . . The main features of the majority of the plans are an embanked roadway on the north side of the river, and the formation of docks with the view to retain all the existing wharves; in others, railways in addition to the roadway and docks have been proposed; whilst in a few, a solid embankment and roadway without either docks or railways have been suggested. Amongst these latter is a plan submitted by Mr. Sheilds, some of whose suggestions appear to us to afford in a greater degree than any of the other designs, the basis upon which an efficient and economical scheme may be founded." Also in the evidence given before the House of Commons Committee on the Thames Embankment Bill of the following year, Sir William Cubitt, the Chairman of the Royal Commission, said, "Mr. Sheilds' plan was then brought out again, and we felt that that did nearly meet what we thought we required;" and other members of the Royal Commission, viz.: Captain Burstall of the Thames Conservancy, Sir Henry Hunt, and the late Mr. M'Clean, M.P., Past-President Inst. C.E., gave evidence in succession to the same effect.

Mr. BALDWIN LATHAM said, having had some experience with

regard to the action of tides in rivers, it appeared to him that many of the remarks in reference to the non-interference of embankments with tidal currents could not be substantiated. In 1862, when the great inundation took place in Marsh-land, arising from the failure of the Middle-Level sluice, he made a series of observations on the tidal flow in the Middle-Level cut, and on and off the inundated lands, and from these experiments he had been able to draw some conclusions as to the effect of tides in embanked rivers. Sir Joseph Bazalgette had stated, that the Thames embankments were popularly supposed to have led to the high tides; against that view he had referred to a number of opinions that the Thames embankments had not affected the height of the tide at all, and other speakers had expressed a similar view. But up to the present time it had not been mathematically demonstrated whether the embankment could, or could not, raise the level of high water in the Thames. Apart from all questions of evidence before Parliamentary committees, or any question of interested motive, he thought it could be shown that the mere alteration of the section of the river would materially affect the height of the tide. With simple geometrical sections for the purpose of facilitating the calculations, a triangle might be taken as representing the section of an unembanked river, and an embanked river might be fairly represented by a parallelogram. Both channels should be of the same area and depth; across the top water level the triangular channel should be, say, 200 feet wide and 10 feet deep, and the rectangular channel at its top water level should be 100 feet and 10 feet deep, both channels having exactly the same sectional area, viz., 1,000 superficial feet. Dealing with 1 foot in length, each section would contain 1,000 cubic feet. The river Thames at London might with propriety be compared to a series of reservoirs filling from apertures represented by the openings of the bridges. In the hypothetical case now taken, the two sections might be compared to reservoirs in the process of filling up. If the head under which the filling up or discharge took place was uniform, both the triangular and the rectangular sections would fill and discharge through equal openings in the same time. But if the head was not uniform throughout the time of filling, the sections which were of identical capacity but of different form would vary considerably. It had been shown by Mr. Redman, in his Paper on the River Thames, that the flow of the tide was not uniform, and that the greatest fall, or head, when the channel was filling up, occurred shortly after low water; whilst when the tide began to flow, it rose in the river to such an

extent that instead of coming to a level it attained a higher level in the upper reaches of the river at London than lower down, say at Sheerness. The increase in the rise of the tide at London over that at Sheerness was the measure of the momentum of the stream, and this was expended in producing elevation, which created a natural check to the flow, or retarded the velocity of the flowing tide, thus tending further to unequalize the head producing discharge. When the tide began to ebb in the Thames the fall was the greatest within half an hour after high water, and it diminished down to the period of low water, low water at London being lower than at Sheerness. Here again there was a fluctuating head, so that dealing with a fluctuating head in a river where the sections were identical, and the quantity of water held by the sections was the same, it took much less time to fill a section of an embanked river than it did to fill the natural bed of the river which was of a more triangular form. Dividing the depth of each of the sections into ten spaces, and assuming that when the channel was filling up in the first division there was a head of 10 feet diminishing by 1 foot of head in each division; and further assuming that each figure was supplied with water through an opening of 1 foot area, and neglecting friction, which it was unnecessary to take into consideration in an hypothetical case, it would be found that the velocity was equal to $\sqrt{2gh}$, where $g = 32\cdot2$, and h was the head; and as the area of the opening was 1 foot, so the theoretical velocity would also equal the theoretical discharge.

TRIANGULAR CHANNEL FILLING UP.

Section of River.	Cubic Contents 1 foot in Length.	Head.	Theoretical Velocity and Discharge.	Time of Filling.
		Feet.	Feet per second.	Seconds.
No. 1	10	10	25·37	0·39
„ 2	30	9	24·07	1·24
„ 3	50	8	22·69	2·20
„ 4	70	7	21·23	3·29
„ 5	90	6	19·65	4·58
„ 6	110	5	17·94	6·13
„ 7	130	4	16·04	8·10
„ 8	150	3	13·89	10·79
„ 9	170	2	11·34	14·99
„ 10	190	1	8·02	23·69
Totals . .	1,000			75·40

RECTANGULAR CHANNEL FILLING UP.

Section of River.	Cubic Contents 1 foot in Length.	Head.	Theoretical Velocity and Discharge.	Time of Discharging.
		Feet.	Feet per second.	Seconds.
No. 1	100	10	25·37	3·94
" 2	100	9	24·07	4·15
" 3	100	8	22·69	4·40
" 4	100	7	21·23	4·71
" 5	100	6	19·65	5·08
" 6	100	5	17·94	5·57
" 7	100	4	16·04	6·23
" 8	100	3	13·89	7·19
" 9	100	2	11·34	8·81
" 10	100	1	8·02	12·46
Totals . .	1,000			62·54

From these figures it would be seen that the triangular channel required 75·4 seconds to be filled, as against 62·54 seconds for the rectangular channel, or 20½ per cent. more favourable for the triangular channel than for the rectangular channel, when viewed as a condition in preventing undue elevation of the tide. For as the rectangular channel, although of the same capacity as the triangular channel, was filled so much sooner, and as the tide would flow for a longer period after it was filled than in the case of the triangular channel, this lengthened period of flow must result in an increased quantity of water entering the river. This would have to be stored somewhere, and as a result, high-water level would be higher in the rectangular channel than in the triangular channel. When the tide ebbed, if it were again assumed that there was a head of 10 feet in the first section, and that the head diminished 1 foot in each section, it would be found that the rectangular channel would discharge the water in exactly the same time as it was filling up, or 62·54 seconds, but that the triangular channel would discharge in 49·7 seconds (see next page); so that the triangular form of section was more favourable for discharging the largest quantity. This had been established by long experience in the Fen district. In dealing with large bodies of water for the purpose of discharge, the water was stored in the washes, the banks being set back from the river, sometimes at a considerable distance. The consequence was that, when there was a large quantity of water in the rivers, the bulk of it was stored at the higher level, or under conditions most favourable for its discharge. The mere alteration of the form of section of a river materially altered its

TRIANGULAR CHANNEL DISCHARGING.

Section of River.	Cubic Contents 1 foot in Length.	Head.	Theoretical Velocity and Discharge.	Time of Discharging.
		Feet.	Feet per second.	Seconds.
No. 1	190	10	25·37	7·48
" 2	170	9	24·07	7·06
" 3	150	8	22·69	6·61
" 4	130	7	21·23	6·12
" 5	110	6	19·65	5·59
" 6	90	5	17·94	5·01
" 7	70	4	16·04	4·36
" 8	50	3	13·89	3·59
" 9	30	2	11·34	2·64
" 10	10	1	8·02	1·24
Totals	1,000			49·70

filling and discharging capacity, when viewed as a reservoir filling or discharging through the openings of the bridges. It should also be borne in mind that, in the rectangular section, the hydraulic mean depth was much greater than in the triangular, the hydraulic mean depth of the triangular section being 4·975 feet, and in the rectangular section 8·333 feet. At 3 inches fall per mile, the velocities per minute would be, according to the formula of Eytelwein, ($55\sqrt{2 H r}$), in the triangular channel 86·729 feet per minute, and in the rectangular channel 112·266 feet per minute; and both channels being of the same area, the discharge in each case would have 1,000 times the velocity, or 86,729 cubic feet per minute for the triangular channel, and 112,266 cubic feet per minute for the rectangular channel, or 29·44 per cent. in favour of the rectangular channel. The discharging capacity of the rectangular channel was greater than the triangular channel; but it must not be lost sight of that the power of filling up was also greater, and the one more than counterbalanced the other, when viewed in reference to the height to which the tides would rise; for whether the section was merely altered to increase the velocity of flow, or the time of filling was shortened, the more rapidly the channel filled, the higher would the tide rise. Again, the greater the velocity with which the tide flowed up the river the greater its momentum; and in this case momentum was expended in lifting the water, and so increasing the height of high water. It had been stated that the same amount of water flowed up to London Bridge now as in former years when there was a larger area of storage. Was it common sense to say that when 23,000,000 cubic feet of storage capacity had been taken out of the river there

could be the same amount of room for the water flowing up? It could hardly be credited that no more water flowed up to London Bridge now than formerly. The dredging and improvements in the channel must have increased to some extent the hydraulic mean depth of the stream, and this in the face of a contracted channel and less storage must add to the height of extreme tides, especially if they occurred at a period of land floods in the river. The embankments of the Thames induced the elevation of the tides from four causes: 1st, by reason of the alteration of the section—the new section being both smaller and more rectangular than the original channel would fill more quickly; 2nd, by reason of the velocity of flow being greater in the smaller and altered channel it would fill more quickly than the original channel; 3rd, by increased momentum of both the tidal flow and land floods which was expended in elevating the water; 4th, by reason of the absolute capacity of the river for the storage of water having been diminished. The embankments had added greatly to the beauty of the metropolis, and reflected credit upon their engineers; but he maintained that they ought to be extended. Those who had suffered from them were entitled to some consideration, and he hoped engineers would give attention to the South side of the river and remove all causes of complaint.

Mr. Henry Law said although two attempts had been made, one upon general and the other upon mathematical principles, to demonstrate that the height of the tide must necessarily have been increased by the construction of the embankments, he had, after careful attention, entirely failed even to understand them. He thought those who persistently advocated the view, that the effect of the embankments had been to raise the height of the tides in the upper reaches of the Thames, rather looked upon the river as though it were a pint pot into which, after having put in a few stones, one still persisted in trying to pour a pint of water, so that of course some of it ran over the top. The truth was that there was no express quantity to be delivered at the mouth of the river. There was a certain amount of tidal force, depending upon varying circumstances, but under the same conditions that force was constant. It had to expend itself in doing work in flowing up the river; it had to expend itself by the friction of the bed and by the resistance offered by artificial obstructions, such as bridges and varying widths. The residue of the force that was not expended in overcoming those resistances was overcome by heaping up the water against gravity. As, therefore, the resistances arising from the

friction of the bed and from the obstructions were diminished—as was done on the removal of such bridges as Old London Bridge, where the sectional area was formerly about 7,360 feet, and at the present time about 17,600 feet, and, in a similar way, Blackfriars and Westminster bridges, and by removing shoals—a larger portion of the tidal force was left unbalanced which could only expend itself by heaping up the water. Again, when by the removal of Old London Bridge the low-water line was lowered nearly 6 feet, it became absolutely necessary for the purposes of navigation that the bed of the river should be lowered by dredging, and the increased quantity of water which came up, and the increased force which it had for scouring tended to lower the bed, and that again diminished the resistance offered to the tidal flow, by making a less incline for the water to run up; just as if the river represented a railway on a rising gradient, the tide representing a train sent on to the incline with a given force, if by any means the friction of the wheels upon the rails was reduced, the train would run to a greater distance up the incline before expending its force. He wished to say one word with regard to the positive effect, mathematically considered, of the Victoria Embankment upon the height of the tide itself. He had calculated the sectional area which the river had at Fife House, close to Whitehall Stairs, before the embankment was formed. Formerly the sectional area there was 24,870 square feet, and the hydraulic mean depth was 19·09 feet, giving a velocity of 1·9 foot per second through that section, and allowing 47·444 cubic feet of water to pass per second. The effect of the embankment was to diminish the sectional area to 21,600 square feet; but at the same time, in consequence of its cutting off a long range of shoal water upon the Middlesex side, it increased the hydraulic mean depth to 22·04 feet, and the velocity to 2·01 feet; therefore, although the sectional area had been diminished, and the velocity increased, the quantity of water flowing up was only 43,490 cubic feet per minute. The double result of that was, first (as far as it was possible to conceive any effect to be produced) to diminish the height below, because it required less force to drive the water through the section, and secondly to diminish the height of the water above, because so much less water had passed in a given time.

Mr. Scott Russell observed that Sir Joseph Bazalgette had, in his remarks, almost completed the discussion of the subject. He was glad it had been explained that there was no defect in the engineering design to cause the altogether exceptional fault in the Chelsea Embankment. Great thanks were due to Sir

Joseph Bazalgette, and to the Board he served, for the noble work that had been accomplished, not only with economy, but with good taste, in which engineers were often supposed to be wanting. He entirely agreed in the opinion that the embankments were in no degree guilty of the faults that had been imputed to them with reference to the increase of floods. He had given great attention to the subject of tides in rivers during the last forty or fifty years. He had seen rivers greatly damaged and greatly improved, according as the engineer meddled or abstained from meddling with the tidal stream. One of the merits of the Victoria Embankment was that the engineers had simply taken Mr. James Walker's natural continuous flow of the lines of the river and embodied them in the work. He did not think that these embankments, all put together, produced any sensible effect on the tides—not more than an inch or two. The cause of the floods coming down the river had been well stated to be the new system of drainage adopted around the whole basin of the Thames, which discharged the water after a sudden fall of rain in half the time formerly occupied. That was enough to inundate the river, and who would imagine for a moment that the Victoria Embankment could invite the river for miles up to come down more quickly than it did before? It was a perfect absurdity, and it was equally absurd to compare the effect of the tidal wave in coming up to anything that the Victoria Embankment could do. The tidal wave was brought up by the force with which it originally entered the river; but it was led up by the wise embankment of the lower part of the Thames, and the dredging and deepening of the navigable channel below had the effect of enormously increasing the velocity with which the tidal wave came upwards. It was that which produced the high tides, and the Victoria Embankment was too short, and too unimportant in narrowing or altering the shape of the river, to affect the enormous mass of water which the tide from Sheerness sent up the channel with a given velocity. That velocity was increased far more by removing the shoals, by embanking the lower part of the river, and by dredging, than by any other cause. He attributed none of the evils that had been spoken of to the Victoria Embankment, but he begged to point out a serious danger in the future. With every progress in the improvement of the river below, there would be higher and more exceptional tides; and with every improvement above in the basin of the Thames, there would be a more rapid delivery of the water in the upper Thames. Every now and then, too, there would be those exceptional tides which arose from the remarkable fact that two tides

entered the Thames instead of one, coming in different ways, one from the west and the other from the north. When it happened that one of the exceptionally high tides was westerly, and the other northerly, the two coinciding with similar high winds, there would be a much higher tide than usual, because the more the banks of the river were smoothed, the more the channel of the lower Thames was deepened, the more were the extraordinarily high waters exaggerated by the facility given to the propagation of the wave up the river. He thought it was important that precautions should be taken for the future, and he was not rash in asserting that, before the young generation of engineers had finished their work, they would find tides coming up the river 2 feet if not 3 feet higher than any they had hitherto had to contend with. He knew rivers in which the tides were several feet higher than they were in his youth. With regard to the Surrey Embankment, he thought it had been most wisely omitted. It could do no good in preventing floods. One side of the river should be devoted to commerce, the other to pleasure. The bridges had no doubt occasioned a good deal of impediment. They first dammed up the fresh water, and secondly dammed up the flood in its rising, and therefore produced what might be called local exaggeration in the tides. The removal of the bridges had only had one effect—the removal of those two irregularities. When the new bridge was made over the Thames, below London Bridge, he hoped the engineer who had conducted the work of the embankment so well would not put further impediments into the river. Modern engineering could span the river at its full breadth with a perfect and economical bridge, which being built of one arch should not intrude on either side upon the waters of the river, or interfere in any way with the navigation of the Thames.

Mr. W. Parkes had listened with great interest to the last two speakers, who so clearly and correctly described the laws under which tide was propagated up a river; but he confessed he was surprised to find that they had arrived at the conclusion that the embankment had no effect upon the height of the tide. The circumstance which regulated the quantity of water entering into the upper reaches of the river was, it appeared to him, the momentum of the great volume of tidal water coming up through the lower reaches from the sea. It was in no way —or only to a slight extent—dependent upon the height to which it rose in the upper portions of the river. It came up past the Pool and past London Bridge with an enormous momentum, and it could not be kept out by any small obstruction in the

river above; but if the space were reduced which it formerly filled, by such objects as the embankments, the water forced up by that momentum must find room for itself somewhere else, and he did not know where it should be except by rising a little. If the 52 acres which had been reclaimed by the embankment occupied space formerly filled by 10 feet of tidal water, multiplying the 52 acres by the 10 feet would give the quantity of tidal water, which was sent up the river as before, but which had to find room for itself somewhere else. If that were spread over the whole area of the river between Blackfriars Bridge and Teddington Lock, it would give a depth of about 4 inches. That, he believed, was something more than the elevation of the tides due to the embankment, because that head of 4 inches exercised a small amount of counter-action to the momentum of the tide. He did not think that the embankments would have the slightest effect upon the land-floods coming down, or in raising the water as far as Blackfriars Bridge; but upwards to Teddington Lock he thought they produced an effect of something less than 4 inches. To say that the embankments were an important cause of the additional flooding of late years was a great exaggeration, though he believed they had helped towards it in a small degree.

Mr. William J. Doherty observed, through the Secretary, that having been engaged in preparing tenders for almost the whole of the works described in the Paper, the particulars of the completed contracts afforded him more than ordinary interest. The accepted tenders amounted to £875,000, and the Author stated that the total cost of the works had been £1,200,000. It would be an advantage to be informed how the additional 37 per cent. on the contract amounts had been absorbed. However, it was mostly in reference to the cofferdams, and the temporary works required in carrying out the works, that the Paper would have for a number of the members the greatest interest. It had been assumed by the Author that the dams formed of wrought-iron caissons were preferable to, and cheaper than, timber dams. From the Paper it would appear that the chief advantage possessed by the caisson dams was in allowing a strengthening toe of concrete, 7 feet thick and 14 feet in height, to be added, by which "the concrete at the toe of the wall, together with the gravelly substratum upon which it rested, was completely enclosed, and a firmer and deep foundation economically obtained." (*Ante*, p. 14).

So far, it could not be upheld that this was a fair mode of comparing the wrought-iron caisson dam with the timber dams. Again, as the land water, in some cases, overpowered the pumps, the concrete was lowered into place through the water, and the foundations of the river wall were not laid dry. This seemed to be the effect of not having sufficient pumping power, and therefore could not be laid against the caisson dams, as the water was land water and not tidal. Comparing the cost of the caisson dams with the timber dams, the Author showed that were it not for the allowance of £8 per lineal foot credited to the contractor on account of the concrete toe, the iron dams would have been more expensive by £6 per lineal foot. It had not been shown that in the matter of time, or superiority in getting the foundations in with less water, that this form of dam had any advantage over the timber dams. Although they could not be regarded as an improvement, he was free to admit that the experiment was an interesting one, and the thanks of the Institution were due to the Author for the particulars he had recorded. Considering the employment of cement concrete, and the moderate depth, not exceeding 14 feet below low water, at which the foundations of the river wall had been laid, it had always been his opinion that the most expeditious, safe, and economical mode of accomplishing this work would have been that of driving two rows of grooved cast-iron sheet piles, with their heads standing up about 4 feet above low water, having timber wales and struts, temporarily attached to keep them asunder, to the proper width of the foundation, as had been so successfully adopted on the Mersey, in putting in river wall foundations, by the late Mr. John B. Hartley, M. Inst. C.E. Powerful pumps would speedily have removed the water, if it were preferred to put in the concrete dry; if not, the excavation could have been dredged, and the concrete lowered through the water, as seemed to have been done in some cases even with the wrought-iron caisson dams. This form of dam would have been put in for about £10 per foot, and if concrete blocks had been used, such as were adopted for the Chelsea Embankment, the price could have been further reduced by the drawing and re-using of at least one-half of the grooved sheet-piles.

Mr. J. W. Johnson remarked, through the Secretary, with reference to the means adopted for sinking the caisson dams for the Victoria Embankment, that in India no engineers would ever have thought of adopting the pneumatic process. The method in India consisted almost universally in undersinking, by removing the material from the inside of the cylinder by an excavator, of

which several forms, adapted to the varying nature of the soil to be dealt with, had been patented. Of these he might mention two, known respectively as Fouracre's and Bull's[1] Excavators. The former of these was used in sinking the brick wells which formed the foundations of the Sone anicut, at the head works of the Sone Irrigation system in Bengal, and no more effective tool for the purpose could well have been designed. In this system engineers in India had only followed the old native plan of sinking wells by the jham, in which improvements had been made, resulting in economy and rapidity of work. He entertained no doubt that, making all allowances for the difference in the cost of labour in England and in India, these caissons would have been sunk much more rapidly, and at much less cost, in India. It certainly seemed strange that between the time when the pneumatic process was invented and the time when this work was carried out, there had been so little improvement in England in the means devised for sinking cylinders under water; whilst in India, within a much shorter period, immense strides had been made in this direction.

Mr. W. Lawford stated, through the Secretary, that the river wall of the West London Extension railway, which was built in 1861–62, at a cost of £15 15*s.* per lineal yard, had never shown the slightest subsidence, although the foundations were 3 or 4 feet above the bed of the river, and heavy coal trucks, &c., were constantly close to the top of the wall. This work was originally between 500 feet and 600 feet in length, but two years ago an additional length had been built, 1,200 feet in length, and of the same section he believed, by the London and North Western Railway Company. With regard to the floods in the Thames, and especially those of last year, he did not think the modern embankments had nothing to do with them.

Mr. Henry Robinson observed, through the Secretary, that in any future extension of the Thames Embankments, it would be well to consider whether there were no means of avoiding the heavy cost incidental to the loss of frontage of the business premises along the river banks. When the Metropolitan Board of Works was considering the question of adopting some measures to prevent a recurrence of the periodical floodings of the low lands at Lambeth and the neighbourhood, the Engineer to the Board reported that "the expenditure on the works, although very large, will be much less than the expenditure for the compensa-

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[1] *Vide* Minutes of Proceedings, Inst. C.E., vol. xxxix., p. 212.

tion for injury done to the business of wharves along the shore, and that expenditure must be counted by millions." In order to obviate this enormous expenditure for compensation, he suggested that future embankments which interfered with wharves, &c., should be constructed with an outer wall following, or nearly so, the river frontage, and an inner wall facing the present river front, the space between the inner wall of the embankment and the present river frontage to be of sufficient width to form a canal communicating with the river by either single gates or locks; and that the embankment should have a footpath and roadway next the river, connected at intervals with the present roads. The necessary protection would thus be afforded to the foreshore, not only without injury to the frontages, but with the further advantage of enabling the barges and other craft to be afloat the whole day, instead of being, as now, stranded during low tide.

Mr. SHOOLBRED remarked, through the Secretary, that reference had been frequently made in the course of the discussion to the action upon the tidal stream of the embankments in the lower reaches of the river Thames, which embankments were of great antiquity, and had been constructed to keep out the tidal water from the low-lying marshes behind them. He wished to point out an example, of recent date, near the mouth of the river Seine, where dykes had been formed in order to deepen and improve the tortuous canal below Quillebœuf, by giving the river a more direct course, and thus rendering the tidal action more effective. These dykes had been continued down as far as Berport, where the wide-mouthed bay began. They were only completed in 1866. The changes already wrought by them were remarkable, and were still continuing. The navigable channel up to Rouen had been deepened and improved, with but little aid from dredging, and without increase in the waterway, by the removal of bridges or obstructions of a similar nature. Furthermore, large areas of slob-lands had gradually been reclaimed, by the deposition of the mud behind the dykes, increasing in proportion in elevation till they had already in most places become pastures above high-water level. But, while in the river the results had been beneficial, in the bay they had been far otherwise. The tidal inflowing stream had always brought with it from the west a large amount of sand, which in times gone by had been carried up the river and deposited gradually at many points during the upward course of the tide. But at present, owing to the narrow mouth presented between the entrance dykes at Berfort to the rapid current existing there, and to the slob lands being piled up with accumulations, the sand-

charged tidal water was obliged to seek other depository ground than formerly, which it found in the adjoining portions of the bay. Accumulations from this cause had been augmented during the last few years to an extent of several million cubic yards, which, if continued, might in time seriously affect the Port of Havre. Another unexpected result of these improvements was, that owing to the much smaller supply of tidal water required to fill the bed of the river, the flow of tidal water round the bay of the Seine had been quickened, and the time of high water on its eastern shore advanced upon what it had been previously. At Havre this acceleration in time amounted at spring tides to nearly forty minutes, while the times of low water remained the same. This example might be considered instructive in the discussion of the effect of embankments in the Thames and upon other rivers. For here the alterations were due almost entirely to embankments, low water ones at first, but which gradually had risen to be high-water ones in spite of efforts to check and reduce them to their original dimensions.

Mr. G. H. Tait drew attention, through the Secretary, to the statement that the caisson dredger, used so successfully, had excavated the work at 8*s.* per cubic yard, as against the other methods at 12*s.* and 14*s.* 6*d.* per cubic yard; and that out of 2,440 lineal feet of caisson dam only 212 feet had been sunk by the cheapest method, that of the dredger; and that the next more expensive method, the pneumatic system, had only been adopted for 187 lineal feet of the dam; whilst 1,962 lineal feet had been sunk by bag and spoon, and by manual labour, in the slowest manner and at the most expensive rate, 14*s.* 6*d.* per cubic yard. There surely must have been some good reason for this, as it seemed impossible that the contractor of his own free will would sink four-fifths of the caissons at 14*s.* 6*d.* per cubic yard, when he had the means of doing them by the most rapid and least costly of the methods employed at 8*s.* per yard. This valuable machine had been invented by the late Mr. James Slater (at that time one of the agents of Mr. Furness), especially for this work. It was patented as No. 1,863 of the 26th of July, 1864, in the joint names of Messrs. George Furness and James Slater, the final specification being dated the 26th of January, 1865. At that time the machine was working well and satisfactorily. The cost of the machine was, he believed, £500 complete; it did its work at an expenditure of 55 per cent. of the methods generally adopted, viz., the bag and spoon, and manual labour.

Mr. Edward Bazalgette said, before replying to the various

questions which had been raised during the discussion, he wished to express his thanks to previous speakers, to those who had at the outset introduced and encouraged a debate on subjects of public interest, and more especially to those who had lightened the burden of his duties by their conclusive answers to questions requiring explanation. Mr. Tait had asked why the telescopic dredger was not more extensively adopted in sinking the caissons on the Victoria Embankment? The reason was that its introduction upon the works was not sufficiently early for a more extensive use. The pneumatic process was chiefly used for getting in the foundations where the nature of the work was difficult. The price given in the Paper for excavating a cubic yard of soil by various processes included labour only, and not the prime cost of the various mechanical contrivances. Mr. Doherty had asked if the contracts for the Victoria Embankment works amounted in all to £875,000, what portion of the work absorbed the extra £325,000 which ultimately raised the total cost of the works to £1,200,000, or an extra 37 per cent. on the contract price. It had not been fully explained in the Paper that the sum of £1,200,000 was the estimated and not the real cost of the embankment works. That sum included several items which were estimated for but were not included in the contract. Thus £60,000 were included for statuary which was never erected; £14,000 for fencing; £51,000 for establishment and wages; £9,200 for the temporary footway by the river; £10,000 for the bridges and waiting-rooms on the different steam-boat piers, and various other minor works too numerous to mention, such as the sinking of bore-holes and laying out the ornamental gardens on the Victoria Embankment, and the extras usually arising during the execution of contract works. Mr. Giles had intimated that he would hardly have had courage to use Portland cement concrete in such concentrated proportions as were adopted in the construction of the embankment works generally. At the Adelphi landing-stairs, the site destined for the erection of Cleopatra's Needle, workmen had been recently employed cutting through the concrete and brickwork, in order to remove an arch over the site of which the obelisk was to stand. The concrete exposed at that point gave very satisfactory evidence of the soundness of the work, and the men engaged, having had practical proof of the toughness of the material, had come to the conclusion that the embankment would, for lasting qualities alone in generations to come, gain as wide-spread a reputation as the needle itself. Mr. Sheilds seemed to consider that the Paper professed to give a detailed history of Thames embankments from the year 1367, and stated that no

mention was made of a scheme suggested by himself for the embankment of the river along a certain portion of its length. He was willing to concede that the Paper did take a very cursory view of a few of what he deemed important facts connected with the history of the Thames from a remote period; but the Paper only professed to give the details of the Thames embankments as designed by, and carried out under the supervision of Sir Joseph Bazalgette. He would, however, draw Mr. Sheilds' attention to the fact that his name was specially mentioned among those of several eminent engineers who originated designs and prepared plans for embanking the same portion of the river. Mr. Abernethy attributed the settlement of a portion of the Chelsea Embankment wall as due rather to the lateral thrust of the earth-filling behind the wall, combined with insufficient depth of the foundations, than to the reasons alleged in the Paper. The observations made by Sir Joseph Bazalgette were, he thought, sufficiently cogent to satisfy the Institution as to the true cause of those settlements, corroborated as they were by the statement of facts given in the Paper. The total weight of the Chelsea embankment wall was about 29,000 lbs. per lineal foot. The total horizontal thrust of the earth-filling behind the wall was about 4,070 lbs. per lineal foot; considering the weight of the wall as unity, and taking the co-efficient of friction for concrete upon gravel to be as low as that of cast iron upon oak, then the total horizontal thrust of the earth-filling behind the wall would be 3½ times too small to cause it to slide. But when it was further taken into consideration that the wall extended several feet into the solid ground, and was protected by sheet-piling, it was improbable, he might almost say impossible, that any movement of the structure could take place for any reason other than the one indicated in the Paper. Mr. Abernethy thought that a timber cofferdam of equal length, consisting of two rows of piling, could have been driven in less time than was occupied in sinking a caisson, about eight and a third days. The telescopic dredger was the most rapid method of excavating, for that would sink a caisson to a depth of 20 feet in an average of five days. Disregarding that fact, the question of the time required for the formation of a timber cofferdam depended mainly on the nature of the substrata through which the piles were driven. By referring to Mr. Ridley's Paper on the Thames Embankment Cofferdams[1] it would be seen that the piles had to be driven through layers of compact sand, overlying a bed of porous gravel, interspersed with

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxxi., p. 3.

fragments of shell; and beneath this stratum, and overlying the clay, was a layer of septaria. So great was the resistance offered by these strata to the driving of piles, that out of the total number of piles first pitched in the formation of that dam one-sixth part failed in driving and had to be removed, piles of harder wood being substituted and driven in their stead. The dam being considered to be complete, on a further examination it was discovered that one-fourth of the total number of piles composing it had failed to penetrate the clay, and they also had to be driven down further. Thus $\frac{5}{12}$ths, or nearly half the total number of piles composing that dam had to be driven a second time, and the time occupied in driving was no doubt more than double that which would have been necessary under ordinary conditions. He therefore thought the caisson process would have been a more expeditious manner of forming the cofferdam. Mr. Ridley also referred to the great detriment caused to the timber by forcing the piles through the hard strata, and stated that the piles after withdrawal were found with their ends bruised into a mass of tangled shreds. No doubt the timbers were more or less injured and shattered throughout their entire length by the severe concussion of driving, and this fact would go far to account for the small value allowed for the timber after the removal of the cofferdams. Mr. Shelford had asked, if the caisson dams were cheaper than the wooden ones, why they were not more extensively employed on the Victoria and other embankment works. The reason might be stated thus:—At the time of the letting of the Victoria Embankment contracts, the contractors were given their choice between the use of wood or iron cofferdams, subject to the approval of the Chief Engineer. Experience having formerly been chiefly in connection with wood, that material was chosen. Great difficulty was experienced, more especially on contract No. 1, where the level of the clay was lower than in the other contracts, in securing timber of sufficient length and in sufficient quantities for the purpose required, at a reasonable cost. Sir Joseph Bazalgette therefore recommended the adoption of the caisson system. Mr. Furness carried out that suggestion, and as it ultimately turned out to be successful, there had been no cause to regret that step. With regard to the other contracts on the Victoria Embankment, no reasonable time was afforded for profiting by the experience gained on contract No. 1, as the three works were all in operation together. At the Albert and Chelsea embankments, reliable foundations were found at a higher level, and a great length of the Albert Embankment wall was constructed behind a dam consisting of

a single row of piles, while the whole of the Chelsea Embankment wall was constructed behind a half-tide cofferdam. No doubt those methods were the best adapted for the particular conditions. An opinion had been expressed as to the advisability of forming an embankment on the southern side of the river, between Westminster and Blackfriars bridges. In an æsthetic or artistic point of view, such an embankment would be highly desirable; the advantages to be gained were, however, stated in the Paper not to be so great as those resulting from the formation of the Victoria Embankment on the northern side. Looking at the shape of the river along that portion of its length, it would be easily understood that the whole scouring and transporting effect of the ebb and flood tides was entirely along the northern, or concave side of the river; while along the southern, or the convex shore, the waters were comparatively slack, and there would be a tendency to shoal. In order, therefore, to maintain a sufficient depth for vessels approaching the Embankment at all stages of the tide, a considerable amount of constant dredging would probably be required. It had been suggested by Mr. Scott Russell that one side of the river ought to be devoted to commercial interests. There seemed no reason why, if the proposed embankments were constructed, facilities equal to those at present existing should not be given for commercial requirements by the construction of draw-docks, landing stages, and works of a similar nature. Several speakers had argued that the effect of the Thames Embankments in increasing the tidal range within the river was paltry and insignificant as compared with other causes, and those causes had been ably demonstrated by Mr. Scott Russell and others. He agreed with the conclusions expressed, that the embankments did not invite or compel the waters from a great distance up the river to flow down with increased velocity, and he might add that neither by coercion, attraction, or invitation, did the embankments accelerate the velocity of the tidal wave up the river. Mr. Redman calculated that the embankments displaced an amount of water equal to 800,000 tons, equivalent to a volume of water extending from Westminster up to Teddington Weir, 500 feet in width and 4 inches in height, and he seemed to infer that because that amount of water was displaced, the same amount of water had continued to flow up the river as formerly, which water would be heaped on the top of the tide to an extra 4 inches in height. It would be more correct to compare the effect of the embankments, both before and after their formation, say, for argument, to

that of two vessels, one double the size of the other, connected by two necks or channels of a sectional area nearly equal to that of the receiving vessel, and supplied with a head of water in both cases alike. In that case one vessel would receive double the amount of water of the other, but the water in both vessels would be at the same level. He knew no law by which the water in the smaller vessel would be piled up double the height of that in the larger vessel, in order to compel it to contain as much water as that contained in the larger vessel, and this seemed to be the effect of Mr. Redman's reasoning. Mr. Law had, in his opinion, condensed the true theory of the whole matter within reasonable limits, and such as would form a fair basis for future scientific investigation.

Mr. Bateman, President, said he was afraid he could not close the discussion without saying a few words, because the subject was one to which he had given a great deal of attention, and he should be sorry if some of the opinions which had been pronounced were allowed to go uncontradicted, or at all events unquestioned, to the public at large, as they had done through some partial and unauthorised reports. Mr. Edward Bazalgette, in the reply he had given to many of the observations elicited by the Paper, had stated that Mr. Scott Russell "demonstrated." That gentleman had asserted, but he did not demonstrate, and he might say with reference to almost every speaker upon the question, that amongst a great deal of assertion there had been no demonstration, or very little. The question was a difficult one. The short title of Mr. Bazalgette's Paper was "The Embankments of the Thames," and it was not therefore at all unnatural that so large a question should have led to a somewhat discursive discussion. It led, of course, to a general consideration of the effect of Thames embankments, and more particularly to the effect of the embankments described in the Paper, and of the removal of the great bridges upon the river. Nothing was so certain in hydraulics as that an enlargement and contraction, one coming after the other, affected the current of the water and the velocity of the stream. Engineers could not admit water into a river through a narrow opening, and afterwards expand it into a wider opening without diminishing the velocity. The sectional area was increased, a larger space had to be filled with water, and to a certain extent, whether a large or a small one, the quantity of water required to fill that space must take away from the quantity of water which passed up the stream. Contract that and build a continuous wall and the water must naturally be raised. The water was already in the river; long before

it attained its maximum height at Westminster it had already gone back to the sea; the water therefore did not come from the sea, it was carried forward by momentum, and nothing could have been more clearly explained than it was by Mr. Law, that the tidal force expended itself in heaping up water against gravity. The consequence was that in all tidal rivers no improvement could be effected by enlarging the sectional area, or removing obstructions to the free flow of the water, without raising the water in the higher parts of the estuary. It was so in the Thames and in the Mersey. The water rose about 2 feet higher in the Mersey at Runcorn than the maximum height at Liverpool, although by the time it had attained its maximum height at Runcorn it had fallen at Liverpool 5 or 6 feet, and the whole body was going back to the sea. It was momentum or tidal force which carried the water forwards; and therefore if the water was excluded from the slob lands, which had been so beneficially covered by Sir Joseph Bazalgette, the water must be raised either directly opposite the embankment, or that quantity of water which would otherwise fill those slob lands must be sent higher up the river. What the effect of the embankments might be it was difficult to say, but he could quite believe that it was nearly or absolutely immaterial. Considering the relative amount due to each of the various causes which operated in carrying the water forward and enabled it to rise to a higher level than formerly, there could be no doubt whatever that the removal of Old London Bridge did materially affect the level of the water. A fair upon the Thames, as when Old London Bridge existed, was no longer possible, because it required still water. It was not still water now. It was then something like a mill pond; the water could not pass through the arches of Old London Bridge; therefore it was impounded to a great extent above bridge, and ice could accumulate, and a fair was held upon its surface. Now, in consequence of the larger space through which the water passed, and the increased velocity consequent upon it, the water was constantly in motion and no freezing of the surface could take place to the extent it formerly did. That cause had also been in operation through the removal of the greater number of arches of Westminster and Old Blackfriars bridges, and to every one of these causes a portion of the extra height to which the river had risen was due, as well as to the floods from the uplands—the tendency of present agricultural improvements being no doubt to increase the flow of water in flood time by the increased drainage of the country. It might possibly be that the amount due to the encroachment upon the

river by the Thames Embankments was small, but that it was nothing he must most emphatically deny. One assertion perhaps might be as good as another—he would not undertake to demonstrate it, but he must assert as the result of a considerable amount of consideration of this particular question, that to some extent, however inappreciable that extent might be, the erection of the Thames Embankments, as described by the Author, had had an influence; it might be that influence was not practically important, but still it had had an influence, and he should be sorry for it to go forth that the opinion of the Institution was, that such an encroachment upon the river had no influence whatever upon the current. He did not wish to say anything beyond that, or to express an opinion upon the effect which might have been produced. He thanked the Author for the clear manner in which he had described the construction of the several embankments, and also for the lucid way in which he had explained many of the points to which exception had been taken.

April 16, 1878.

JOHN FREDERIC BATEMAN, F.R.SS. L. and E., President, in the Chair.

The discussion upon the Paper, No. 1,561, "The Victoria, Albert, and Chelsea Embankments of the River Thames," by Mr. E. Bazalgette, occupied the whole evening.

In accordance with the notice on the card of the meetings, it was resolved to adjourn for a fortnight, in order to avoid holding a meeting on the evening of Easter Tuesday, April 23rd.

April 30, 1878.

JOHN FREDERIC BATEMAN, F.R.SS. L. & E., President, in the Chair.

No. 1,498.—"The Ravi Bridge, Punjab Northern State Railway."

By ROBERT TREFUSIS MALLET, M. Inst. C.E.[1]

THE bridge which carries the Punjab Northern State railway over the river Ravi at Lahore is of thirty-three spans of 90 feet in the clear, and 97 feet 6 inches from centre to centre of the piers. The piers are of brickwork, each founded on three brick cylinders sunk 70 feet below the lowest water-level, of 12 feet 6 inches external and 6 feet internal diameter. Eight vertical tie-bars, $1\frac{1}{4}$ inch in diameter, connected at intervals of 12 feet 6 inches by flat horizontal bars, are built into the brickwork.

The piers are protected from scour by concrete blocks thrown round them. The quantity ordered for each pier was fifteen thousand blocks, or 30,000 cubic feet of solid concrete. Dredging the river bed to receive the blocks to more than 2 or 3 feet below low water would have been difficult and costly, and had they all been put in at once, at this level, they would have formed a heap reaching up to the girders and 65 feet wide at the base, occupying more than half the waterway. The narrow portion of uncovered sandy bottom in the centre of each span would then have been scoured out to a great depth, and into this hole many of the blocks would have fallen. To prevent this, 14,000 cubic feet of blocks only were deposited round each pier in the first instance; the remainder have been or will be added as the river scour makes room for them below water-level. At the ends of the bridge (Plate 3, Figs. 1, 2, 3, 4) the blocks were laid over the whole surface, forming aprons extending out to and around the second pier from each abutment. This apron increased in width and thickness towards the abutments, and extended round their foundation cylinders, and for 100 yards backwards, on and in front of the slopes of the approach embankments. The site for the apron was dredged to a depth of 12 feet below low-water level at the outer end, sloping gradually up to low-water level at the abutment. Above this level

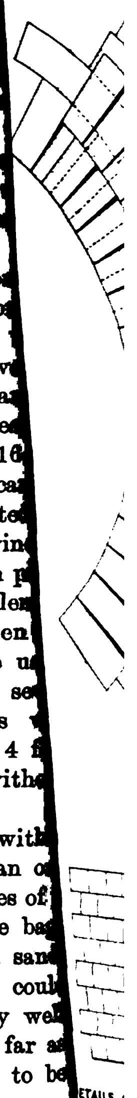
[1] The discussion upon this Paper was taken together with that upon the two Papers following and occupied portions of two evenings.

the slopes of the embankments were pitched with blocks re
laid, and over them with brickwork on edge. By these a
ments the effective waterway at the second pier from each ab
was the same as at the other piers in the bridge, and from
it gradually diminished to nothing at the abutment face.
centration of the scour at any point was thus sought
avoided.[1] The bridge was intended for a level crossing,
used for road and for railway traffic alternately; but the
roadway has not yet been executed.

The girders are of the parallel-flange type. The latti
form two series of triangles inclined at 45°. The superstr
was designed to carry a footpath on the lower flange, a
asphalt cart road, flush with the railway, on the top. The
girders carrying the railway are suspended in stirrups fro
upper flange. The design is identical with that fo
Jhelum.

The first operation was the erection of a temporary w
trestle bridge, consisting of one hundred and twelve spa
14 and 16 feet. The longest timber procurable for the pile
18 feet. The piles were 12 inches square, and were driven 16
into the clean sand, leaving a length of 2 feet projecting for sca
to the uprights. Ordinary hand piling engines were mounte
tram wheels on rails outside the lines of piles, and steam win
were placed on them. Each engine could drive in a day ten p
or, say, 160 cubic feet. Every pile had each foot of its len
marked on one face as a guarantee for its being driven
full extent without the top being cut off. No shoes were u
The entire bridge, 1,600 feet in length, was erected in se
weeks. The greatest load on one pier of two piles
16 tons. It was found that, if the river left a length of 4 f
of each pile in the ground, the pier would carry this load with
settlement.

During the dry season the scour was, therefore, limited to with
6 feet of the points of the piles by sandbags. On more than o
occasion the scour actually reached the points, and the piles of
pier were left hanging to the superstructure; but when the ba
were thrown in, the interstices became rapidly filled with san
and the traffic was resumed in a few hours. This practice coul
not be recommended for passenger traffic, but it did very we
for material trains for four years. The timbers were, as far as
possible, cut to multiple dimensions of the sleepers, so as to be



[1] This arrangement is Mr. Molesworth's.

Fig: 5.

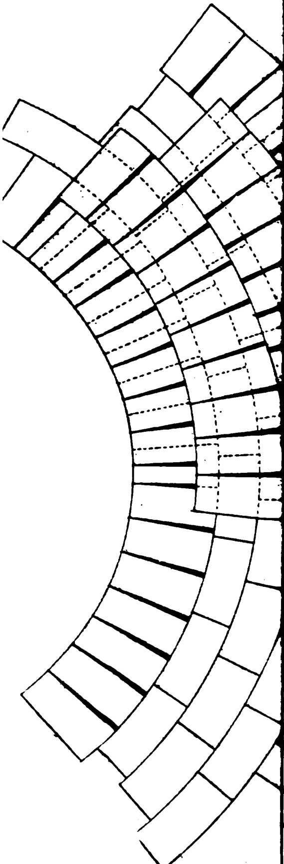
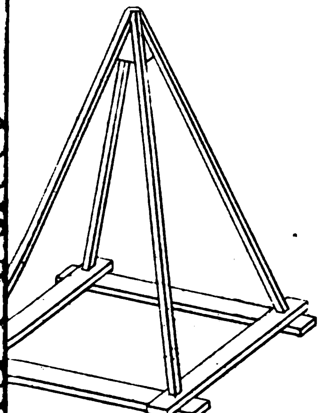
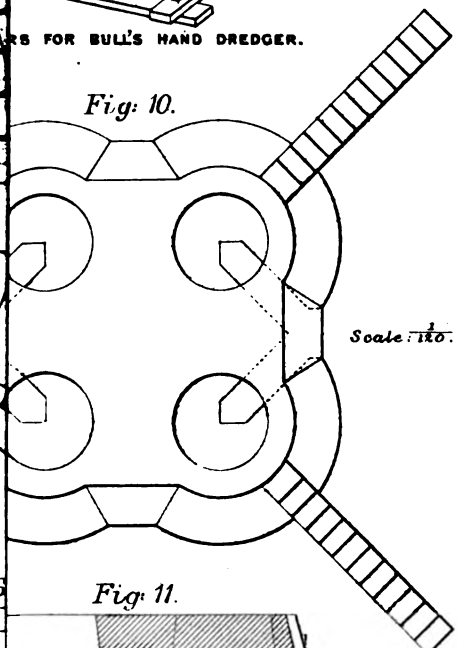


Fig: 9.



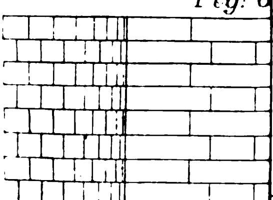
RS FOR BULL'S HAND DREDGER.

Fig: 10.



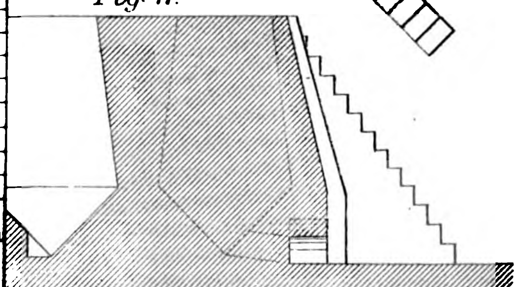
Scale : 1/120.

Fig: 6



ETAILS OF BRICKWORK IN FOU

Fig: 11.



LIME KILN.

convertible without loss when the bridge was no longer required. Large sections of the bridge were scoured out every flood season, but the timbers having been previously roped together, and anchored, were always recovered and put up again. The rails were removed when floods were expected.

The bricks for the foundation cylinders (Figs. 5 and 6) were made of three special forms, viz. a header, laid with its longest dimension radially, a stretcher, laid with its longest dimension circumferentially, and a face header three-quarters the length of the others. The headers and stretchers were about 11 inches long by 5½ inches wide by 3 inches deep, and the three-quarter bricks about 8 inches long by 5½ inches wide and 3 inches deep. The angular divergence of the ends and the radius of curvature of the sides were of the mean radius of the steining. This gave close joints inside and outside. Subsequent experience showed that these bricks were too large, and that similar forms of equal weight with the ordinary 9-inch brick would have been more economical. The special bricks were arranged in two alternate courses—one of all headers on the two faces of the circular wall, with three rings of stretchers between; the other with three-quarter bricks on the two faces, and two rings of headers between. In such circular brickwork diagonal courses are impracticable, and the only expedient for obtaining good bond, vertically, radially, and circumferentially, is the use of three-quarter bricks.

The lime was slightly hydraulic. The kunkur of the district was mixed with one-fourth of its bulk of hard wood charcoal and burned in circular kilns (Figs. 10 and 11) of rather greater diameter at the bottom than at the top, with the bottom sloping inwards to a point. They were filled up twice a day with the mixed kunkur and charcoal, and about 20 cubic feet of lime were drawn twice a day from each kiln. The capacity of each kiln was about 350 cubic feet. The lime was thus about nine days passing through the kiln.

The lime was filled into wooden frames on a brick floor, and struck off to an even depth of 4½ inches, and the exact quantity of water required for slaking was added while the lime was still hot from the kiln. It was then beaten with wooden rammers and screened. It was found more economical to use the screenings for ballast for the permanent way than to re-burn them; also that the screenings could not be made into good concrete, as the semi-vitrified surface of the particles prevented adhesion.

The soorkee or brick dust for the mortar was ground in mills, designed by Messrs. Marillier and Edwards of Calcutta, consisting

of a pair of fluted rollers above and a smooth pair below, each pair being held together with powerful springs. The wear of the bushings was very great, and their repair tedious and costly, as the whole machine had to be taken asunder for the purpose. The mill consumed 8 HP., and the out-turn could not be raised above 400 cubic feet of brick dust in twenty-four hours. Ordinary pan mortar mills were also employed occasionally for grinding up the bricks. Probably the disintegrating machines so largely used for other materials would give good results, but the Author has not been able to ascertain what is their actual performance in cubic feet an hour. A mill is required for large works that would yield 1,000 cubic feet of brick dust a day, and another of a portable form to give 300 or 400 cubic feet for scattered works. A coolie can only pound about 3 cubic feet a day, and the supply of large quantities in a limited time is sometimes a serious difficulty.

The mortar was composed of equal volumes of lime, soorkee, and fine river sand. It was ground and mixed in the ordinary pan mills with two edge runners. Each mill was fitted with a measuring box, divided into three compartments for the dry materials, and with another box for the water. These were filled while the mill was grinding the previous charge. The entire quantity of water and lime for one charge was first filled into the pan and ground to a thin cream. The brick dust, which had already been ground, was then added, and finally the sand, which required no grinding. A few more turns of the mill sufficed to mix all together. By these means each 6-foot mill could be charged and emptied fourteen times in a day of eighteen hours, with economy of steam power, since the lime, of which the hard particles required the most grinding, was not protected from the action of the rollers by the soorkee and the sand. Three pan mills supplied mortar for 100,000 cubic feet of brickwork a month.

The bricks were carried by rail to the bridge. The amount of breakage was 4 per cent. The mortar in the brickwork set well, either in air or water, in two or three days. Similar mortar in the concrete with which the cylinders were filled, by lowering it through the water, did not set after four months: yet this same concrete, if filled into a tub and immersed in water, set perfectly in a few days. The concrete was made of broken waste brick, and 42 per cent., by measure, of the same mixture as the mortar for the brickwork, but not ground. The skips for lowering it were suspended from a self-disengaging claw, like that of a piling monkey and could not be discharged until they had touched the bottom.

The concrete blocks for protecting the piers from scour were composed of

Broken brick	. . . . . .	100 cubic feet;
Kunkur lime	. . . .	16 " "
Brick dust	. . . .	13 " "
Sand	. . . .	13 " "

making 100 cubic feet of concrete.

The materials were spread in layers in large stalls and mixed, once dry and once wet, by hand, along the open front of the stall, as fast as required. The concrete was then carried to the moulders and rammed into cubical iron moulding boxes without either top or bottom, holding 2 cubic feet each. The boxes were slightly tapered to facilitate their being lifted off the rammed block. The ramming at once gave the concrete consistence enough to enable it to retain its form after the removal of the box, and a small number of boxes therefore sufficed. One man filled and rammed thirty blocks a day. The blocks were made on the banks of the river, and carried to the piers. Strong concrete could not be made with the lime of the district, or at a moderate cost; hence it was considered that a cube of 15 inches was the largest size that could be conveniently carried by two men, and be capable of bearing falls and shocks without breaking up, while it seemed sufficiently large to withstand the force of rapid currents of water. Subsequent experience showed that in one exceptional case, with a velocity of probably not less than 10 feet a second, these blocks were moved from a sandy bottom on to a level brick floor protecting a bridge, when they were not moved farther, although exposed to a more violent current. The extreme surface velocity of the river Ravi was 9 feet a second, and as the velocity of eddy at the bottom of the deep holes usually scoured round the piers of bridges in such rivers is probably much less than that at the surface, it would appear that there would have been no gain in making the blocks so large as to necessitate their being constructed *in situ*, or being carried and deposited by machinery.

The dredging for the foundation cylinders was effected principally with Bull's self-acting dredgers.[1] The hand dredger with a capacity of about 2 cubic feet was preferred to the large machine with a capacity of 25 cubic feet. They were manipulated by two men on the top of the cylinder, and were hoisted by six men and a pair of bullocks. When the dredger arrived at the top, the bullocks

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxxix., p. 212.

were unhitched and walked back for a fresh lift, while the dredger was being emptied, set, and lowered by the six men. An improvement was made in the locking bolt by inverting its action and hanging it to one side of the dredger. It then disengaged itself by gravity on reaching the bottom.[1] The hoisting shears consisted of a light frame (Fig. 9), which was easily and quickly lifted off and on the cylinder, and only required to be lashed to a few rails to be ready for work. When the large dredger is working under the most favourable circumstances, and in clean sand, it probably lifts more stuff in an hour than the small-sized tool; but it is so liable to come up nearly empty, and the time thus lost before a remedy can be applied is so great, that it is beaten in the long run by the small hand dredger. The small dredger can act at any point round the inside of the wall, while the large one will only act near the centre; in case of a sudden influx of soil—a common occurrence—burying it deeply, it can be temporarily abandoned and dredged down upon with another, while a similar accident with the large machine would cause delay. It possesses an important advantage over the sand pump, Milroy's excavator, and all heavy machines, in saving the cost of providing, and the delay of removing and replacing, the heavy lifting and emptying apparatus every time a fresh length of brickwork is added. The weighting, if any, has also to be removed and replaced when building is to be continued; hence it is of importance to add as much brickwork as possible each time. This has the further advantage of adding so much weighting without cost. The drawback of having to lift the dredger and its contents to the greater height sooner than would be otherwise necessary is small in comparison. The practice at the Ravi was to build a cylinder of brick 12 feet 6 inches in diameter and of the same height, and sink it from 10 to 12 feet. Another length of 12 feet 6 inches, total 25 feet, was then added, and sunk about 20 feet; next a length of 25 feet of cylinder, total 50 feet, was built and sunk, as far as possible without weighting, usually 35 to 40 feet. Finally a length of 20 feet was built, completing the 70 feet of cylinder, which was sunk without further weighting, sometimes to 60 feet, when a load of 150 tons of rails commonly sufficed to complete the sinking to 70 feet. The greatest load required for any well was 250 tons. Thus the lifting shears for the dredger had only to be erected and removed four times for building, and

[1] This improvement, as well as many other improvements in well-sinking, is due to Mr. W. Harvey, Assoc. Inst. C.E., then assistant engineer in charge.—R. T. M.

once for loading, and the height to which the weighting had to be lifted on to the cylinder was reduced to a minimum.

Seams of loose kunkur mixed with sand, and of tough silt or clay, were met with, and caused delay. If a single nodule of kunkur was gripped in the lips of the dredger, it prevented them closing tight, and then the greater part of the load of sand was usually washed out while being lifted through the water. Under such circumstances the sand pump often proved the best tool.

A screw of 9 inches diameter was fixed on a shaft of $2\frac{1}{4}$-inch gas tubing, and this, when repeatedly screwed into the clay and hauled out with tackle from above, proved tolerably efficient in loosening it. A better tool was that devised by Mr. Fouracres, and termed by him a spider (Figs. 7 and 8). It consists of six picks arranged round a central shaft, to which they are hinged. A heavy ram, sliding on the shaft, and worked from above by a rope, strikes on a boss, which also slides on the shaft. Six connecting rods, hinged to this boss and to the outer ends of the picks, transmit the blows of the ram. The tool is lowered by a rope attached to the boss, and thus the picks are kept open. It is hauled out of the clay by a chain attached to the shaft. It sometimes retains hold of a large fragment, which can then be at once hauled up, but the ordinary use of the tool is only to loosen the silt, which is commonly reduced to mud, to be removed by the dredger. Nearly all the sinking was done by task work, or petty contract, at rates progressively increasing with the depth. The usual rates for 100 cubic feet were from—

Feet.		Feet.		Rupees.
0	to	10	deep	1·25
10	„	20	„	2·00
20	„	30	„	3·00
30	„	40	„	4·00
40	„	50	„	5·00
50	„	60	„	6·00
60	„	70	„	7·50

The measurement was that of the solid cylinder, 12 feet 6 inches in diameter, or 122·7 cubic feet for every foot penetrated by the cylinder. When kunkur or clay caused much difficulty, daily labour was substituted.

The loading, and unloading, the cylinders was effected entirely by manual labour, by hauling the rails up and down a pair of rails laid at a steep incline. The rails were 8 yards long, some 60 lbs., some 40 lbs. to the yard. Twelve men could load one hundred and twenty a day of the heavier or one hundred and fifty of the lighter rails, say 35 tons, 25 feet high, at a cost of $3\frac{1}{2}$ rupees.

Steam hoisting machinery was available, but was not so cheap, quick, or safe.

Whenever a cylinder showed a tendency to leave the perpendicular, it was promptly corrected by shoring from the ground, and by passing a chain round, the two ends of which were anchored taut at a distance. Rails laid across the chain produced an efficient horizontal pull.

When the wells had been sunk to their full depth, the inequalities in the excavation were filled up level to the top of the wooden curb, or the bottom of the brickwork, with clean sand, so as to form a flat base for the entire cylinder when filled in. The only serious difficulty arose from some trees, at a depth of 20 feet, at No. 18 pier. Sheet piling was driven round the site, and shored from the cylinders, and the inclosed space was dredged out. The trunks were then weakened inside and outside the well, close to the curb, by being bored full of holes with a long screw auger worked from the surface, the point having been inserted by a diver. Chains were then passed under by means of curved needles, and the wood was torn out by powerful lifting tackle. These and previous unsuccessful attempts occupied more than three months, and the pier could not be completed before the rainy season. One and sometimes two divers were constantly at work. The central cylinder of the three forming the pier had therefore to be taken up by blasting the brickwork and lifting the curb. The other two cylinders were then sunk to their full depth, and the sheet piling was drawn before the river rose. After the floods had subsided, an iron cylinder was sunk down on the remaining tree, which was removed like the others, and the iron cylinder drawn by screw-jacks. The brick cylinder was then recommenced, and sunk without difficulty. At the commencement of the works the curbs were pitched 6 inches apart, and much difficulty was experienced from the cylinders drawing together. This was effectually remedied by pitching the remainder 2 feet apart. When cylinders have to be sunk to great depths, they should probably not be nearer together than one-sixth of their diameter, 2 feet being the minimum distance. One hundred and two wells, 12 feet 6 inches in diameter, twenty-four of 10 feet in diameter for the abutments, and about 500 lineal feet of other wells, afterwards abandoned, were sunk 70 feet in two working seasons. The average progress with the large cylinders was 2 feet a day of actual sinking, or 8 inches a day of actual work, including building and loading.

Orders to erect the ironwork were received rather late in the

season, and all resources were employed in getting the main girders riveted together and lifted into place before the flood season in June, so as to be able to continue work on the cross girders, &c., during the floods. Half the girders were put together on the south bank and half on the north, and they were carried into place by two travelling gantry cranes. Spans 1 to 9 were over a high sandbank, only covered during high floods, and were left to the last. Spans 9 to 18 were over the main channel of the river, and the girders for them were floated into place.

Spans 18 to 33 were over low sandbanks containing one minor channel, dry until late in the season. The rails for the craneways were laid over the sandbanks, and the channel was provided for by leaving spans 25 and 26 open, and carrying the craneways over the channel on piled staging. These arrangements proved to be sufficient; but a rise in the river rendered the craneways impassable a few days after they had served their purpose.

The floating of the girders for spans 9 to 18 was accomplished on four of the weak native barges, by lashing them together in two pairs, and distributing the weight carefully over the bottom. Timber staging, 16 feet high, was erected on each pair of boats. The remaining height required to enable the girders to clear the piers was made up by a stack of cross sleepers. This provided for the changes of level in the river during the operations. Piled staging was erected to carry the crane out to pier 9, the first pier that was fairly in the water. The girder, which weighed less than 12 tons, was carried out to this pier, and was then lowered until the outer end rested on the staging in one pair of barges, and a point a little behind the centre rested on pier 9. The crane was then run back to lift the back end of the girder, when the whole was moved forward, the second pair of barges was brought under, and the girder lowered on to them. Screw-jacks and slacking-wedges enabled each girder to be lowered into its place in a few minutes. The girders were lifted, conveyed 400 feet, transferred to the boats, and thence to their respective piers, at the rate of five in two days. On the south side they were lifted and moved from 200 to 1,200 feet at the rate of three or four a day.

In each case the compression flange was riveted complete before the tension flange or the lattice bars were put together. This secured closer joints than could be obtained by bolting all together before beginning the riveting. The rivets were nearly all $\frac{7}{8}$ inch in diameter, and were closed by task work, at the rate of 5 rupees a hundred, by men who had most of them already had experience of such work on the bridges of the Delhi railway.

The rate of progress of the Ravi bridge was too much affected by official causes, distinct from the struggle with Nature, for details to be of value for purposes of comparison.

The following table gives the actual cost of the principal items of the work, excluding the cost of plant, and of engineering and supervision:—

——	Unit.	Rupees.	Sterling at 1s. 9d. per Rupee.
			£. s. d.
Wood fuel	Ton	7	0 12 3
Lime slaked and screened	100 cubic feet	29	2 10 9
Soorkee, or fine brick dust	,,	7	0 12 3
Bricks 11 ins. by 5½ ins. by 3 ins.	1,000	22	1 18 6
,, 10 ,, 5 ,, 3 ,,	,,	20	1 15 0
Concrete blocks 1 ft. 2½ ins. square	100	30	2 12 6
,, ,, in random mass in place	100 cubic feet	10·15	0 17 9
,, ,, net solid contents	,,	15	1 6 3
Concrete filling in foundations	,,	9·88	0 17 3
Brickwork	,,	31·88	2 15 9
Ashlar masonry, heavy plain	Cubic foot	3·19	0 5 8
Open dredging to 12 feet deep	100 cubic feet	4·31	0 7 7
Well sinking to 20 feet deep	,,	6·69	0 11 8
,, ,, 70 ,, ,,	,,	12·06	1 1 0
Girders 90 feet clear span delivered at Lahore	Ton	226	19 15 7
Erection of ditto, including all timber and labour	,,	124	9 17 0
Road metalling	100 cubic feet	30	2 12 6
Earthwork	,,	2·75	0 4 10
Entire cost of bridge	Lineal foot	435	38 0 0

The Paper is illustrated by diagrams, from which Plate 3 has been compiled.

No. 1,502.—"The Alexandra Bridge, Punjab Northern State Railway." By Henry Lambert.[1]

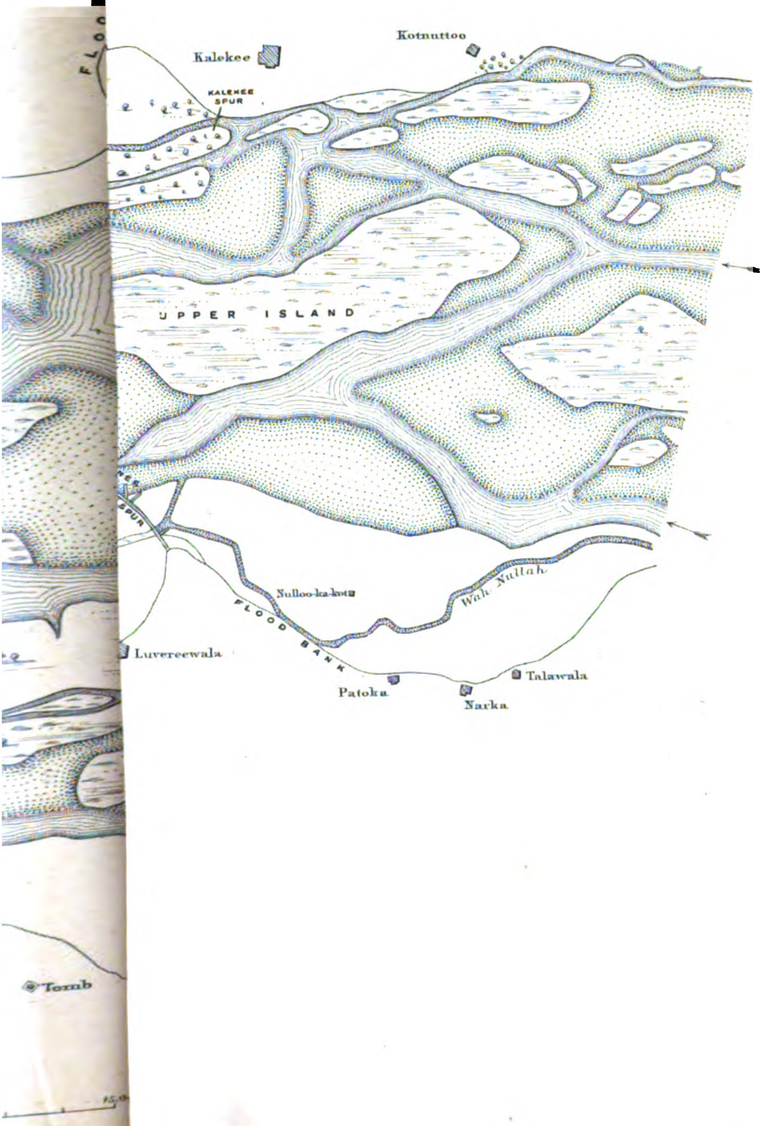
The Alexandra bridge, constructed near the town of Wuzerabad to carry the Punjab Northern State railway over the river Chenab (ancient Ascesines), is 9,300 feet long and 100 feet deep. The first brick was laid on the 1st of November 1871; and the structure was formally opened by his Royal Highness the Prince of Wales on the 27th of January, 1876. The line, which was the first attempted under the new State railway system, is to run from Lahore to Peshawur, 270 miles, and the first section, viz., from Lahore to Jhelum, 103 miles, has been opened for traffic.

The Chenab rises in the Himalayas, and reaches Wuzerabad after a course of about 700 miles (Plate 4). At a point 400 miles lower down, it flows into the Indus, after having received the waters of the Sutlej, Ravi, and Jhelum. Its course for the greater part is through a plain, composed mainly of silt carried down from the high lands during the course of ages. The bed is of fine sand, and the banks are mostly undefined. The river wanders unchecked through the plain, the banks falling in before its advance as the current happens to set. Villages constantly disappear along one shore, while fresh land forms on the other, and again the proceeding is reversed. Trees are found under the cornfields and among the villages, sunk deep in the underlying stratum of river sand, showing where the stream extended in former days. Numerous islands exist, some being part of the general country around which channels have cut their way, and others of more recent formation in various stages of development, and of various sizes, from wooded and cultivated areas of several square miles each, down to small sandbanks. During the winter the river is contained in a main channel about 500 yards wide and 10 to 15 feet deep, in which the water generally flows diagonally to the normal course of the stream, and consequently undermines the banks at the points where it impinges. From this main channel several minor ones branch, and again return thereto. The direction of the current is determined by the promontories, the islands, and the sandbanks. By the middle of April, floods from the melting of the snow on the Himalayas set in, and increase in volume up to the middle of June,

[1] The discussion upon this Paper was taken together with that upon the Paper preceding and the one following, and occupied portions of two evenings.

when the monsoon commences, and lasts until the middle September. Vast floods during this time pour down into t plains, sweeping away the river banks, destroying villages, a scouring out the shifting sand of the bed of the river to a gre depth. In floods, the Chenab rises 11 feet above low-water ma and its width at Wuzerabad is $3\frac{1}{2}$ miles. The mass of the wat does not flow rapidly, but the main stream, corresponding to th fluctuating deep channel, rushes through with great velocity, i a serpentine direction. This is often nearly at right angles t the general course of the river when obstacles occur. Unde these circumstances the bed is driven before it, and the depth o the main current of water is more than 50 feet, moving at a rat exceeding 10 miles an hour. The ground for about half the spac of $3\frac{1}{2}$ miles between the banks is composed of river deposit there being a depth of 2 feet of light soil over fine sand. O this, a massive embankment raised above flood-level had been con structed several years previously, to carry the Grand Trunk Road a far as possible across the river bed, the rest of the way being ove bridges of boats at the varying channels, connected by temporar roads of timber and fascines laid on the sandbanks. These bridge and roads lasted, though with constant interruptions, for abou eight months in each year. During the remaining four months, ferry was established, which frequently necessitated a day's voyag in order to cross from shore to shore. During high floods the fer was suspended altogether.

A back channel, called the Pulkoo Nullah, flows for sever miles parallel to the main river, and in high floods their unit waters submerge the intervening country. The Pulkoo flows pa the town of Wuzerabad, and regains the main stream a few mil lower down. About 3 miles above Wuzerabad, a branch chann 80 feet wide had cut its way through the lowland, and flowed in the Pulkoo opposite the town. The effect of constructing th massive embankment was to dam up the body of comparativel still water forming the bulk of the river, and to divert the swif moving diagonal current against the crumbling shores. Hence th Pulkoo developed into a deep channel 800 feet wide, and becam a navigable stream, enabling large boats to approach the town. A Kathala, on the northern shore of the Chenab, the river becam wider by $\frac{1}{2}$ mile. The main current set against the proposed sit of the northern abutment of the bridge, and the deepest water wa close under the nearly vertical bank. The Trunk Road was graduall falling into the river, while houses and trees along the bank wer constantly carried away; and it became necessary to establish



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ferry at the Pulkoo as at the main stream. Above Wuzerabad, the river pursues a tortuous course; the villages are mostly built away from the stream, and there are no old trees near the river. It is thus apparent that, in the Wuzerabad Reach, the Chenab had become wide to an abnormal extent. The fine sand of the bed was ascertained to be about 65 feet in depth, overlying clay of moderate consistency. With this was present a rapid current moving from shore to shore, and likely not merely to attack bridge piers in flank, where they are weakest, but also to scour the ground from under them.

In January 1870, land was taken and the preparation of materials, &c., was commenced. In February the survey was completed. The main works proposed to improve the site were four in number, shown by the letters T U, P X, M N, and Q R. The first was to close the Wuzerabad navigable channel at its head near the village of Luvereewala, 3 miles above the site of the bridge. This work was ¾ mile in length, and was a massive embankment raised above highest flood-level, cased with bags of earth to resist the action of the water. The ground on which the embankment stood was inclosed at some distance from the toes of the slopes by a double row of piling. Parallel to the river front, trees were thrown into the stream, and securely moored with cables, composed of river grass attached to nets of rope prepared from the same material, and filled with stone. Running out from this breastwork, at an angle of 45°, groynes of a similar character were constructed at short intervals. The river water being heavily charged with silt in times of flood, the foreshore gradually rose under the influence of these works, and as land was thus formed, by encouraging vegetation, and by the introduction of aquatic plants, the interlacing roots formed a barrier against the river. When any portion of the work became scoured out, fresh material was thrown in, until at last a solid shore formed with a gentle slope outwards, and with a curve calculated to divert the main current away from the Wuzerabad channel, towards the opposite shore, near the village of Pindee.

All the islands in the river are below the level of high floods; consequently the water at such times could get round the point U, and again pour into the Wuzerabad channel, wearing the banks in the process, and sweeping away the land remaining between it and the main stream.

The second main work extended from the southern abutment of the bridge to the bank of the Pulkoo Nullah, passing by the village of Hurrypore, parallel to the head work just described.

Its length was about 3 miles, and it consisted of an embankment with a slope of 10 to 1 on the river front, and of 5 to 1 at the back, raised 3 feet above flood-level. At the crossing of the Wuzerabad channel, it was about 100 yards wide at the base, and was protected by piling. The ground in front for 100 yards in width was planted, and several groynes were constructed at right angles, to check sidelong scour. This led to the silt suspended in the overflow water between the two banks being caught in the old channel thus closed at either end, so that the bed gradually rose to the level of the adjacent low ground, and ceased to be a source of danger from enlarging to such extent as to admit the Chenab into its old channel close to Wuzerabad. It was proposed to protect the lower end of the work with masonry at P m, but this design was afterwards altered to a loose filling of trees and stone, supplemented by groynes of the same material, as being more economical. As it seemed unlikely that the main current could be trained in a direct line unless at a prohibitive cost, the river was diverted at the site of the bridge, as nearly as possible at right angles to the general alignment of the railway, by making the current impinge near the point N by a third work, M N. This deflected the stream towards the southern shore, in a direct line through the bridge, and swept away the central island on the point S. This island occupied a position dangerous to the proposed bridge, its tendency being to prevent the direct flow of the main stream, which was somewhat confined by the works themselves.

As the third work M N was intended to receive the full force of the stream turned on to it from the work T U, it was constructed of large masses of trees weighted with stone, and inclosed in heavy piling, about ½ mile in length, set out on a curve, with a versed sine of 350 feet.

The fourth main work, Q R, was proposed to be of masonry, adjoining the north abutment of the bridge, and curving inland, corresponding with the similar work on the southern shore. It was subsequently altered to a star-shaped spur of trees and stones. The object of the suggested masonry works was to prevent the river from cutting behind the abutments in the event of disaster to the up-stream works.

In addition to these main works, several subsidiary ones were executed of rough tree spurs, placed to catch the floating sand, and to assist in turning the stream towards the centre of the river. A commencement was made by laying the ...ee spur, to check the ravages of the current at the site of proposed northern abutment. The flood season coming on

shortly after, not much progress was made until the following October.

The Chenab training works were carried out by September 1872, at a cost of $4\frac{1}{2}$ lakhs of rupees; since then they have been maintained at a further yearly outlay of $\frac{1}{2}$ lakh. To make the works permanent by a covering of stone would probably cost more than three times the original outlay. The future course to be followed in this matter is still undecided, but the completion of the line to the Jhelum river, where boulders are easily procurable, will afford increased facilities. The spurs were formed by launching trees into the river from barges, and lashing them securely by 9-inch cables of river grass, called "moonj." At short intervals weights were attached, consisting of boulders inclosed in nets of 3-inch moonj rope. As the mass of material sank into the sand more was placed above until the object was attained. The country being thinly wooded, considerable difficulty was experienced in procuring suitable trees; and in the vast alluvial plain through which the Chenab flows there is not a stone to be found. The boulders had consequently to be brought from the rapids, 60 miles up stream, beyond British territory. This latter circumstance added to the difficulty.

Boats had to be brought from distant places, and many were wrecked in the service, which increased the difficulty in procuring trees for timber works. Much difficulty was experienced in closing the head work T U. The channel was scoured to a depth of 25 feet. Several large boats were loaded and scuttled on the site to check the current; but they soon broke up. The embankment was at last carried across the opening by hurling in trees and bags, afterwards inclosed by heavy piling; the work being carried on night and day by several thousand men. The bed of the channel rose as expected, and land formed, which was planted and gradually became covered with dense jungle. The work P X was carried across the Wuzerabad channel after the latter had been closed at T U. At the two crossings the widened base of the embankment was inclosed between rows of 30-foot piles, 12 inches square. As channel No. II. contained a large body of water moving at a high velocity, the site was prepared by tree spurs *n o*, *p q*, which caused a gradual silting up of the channel. The curved portion of the main embankment was formed in 1875, with material removed from the central island by ballast trains. Meanwhile only the part marked *m* X was constructed; and an auxiliary embankment *l m* was formed to prevent flood water from pouring into the Pulkoo. Tree spurs were laid down in front of the more

exposed portion of the main embankment when the time had arrived for its economical construction. This was determined when the action of the spurs above had caused the partial silting up of No. II. channel.

The third main work, on the northern shore, consumed a vast mass of material. During the first season, piles 30 feet in length and 12 inches square were driven into the river bed, and trees were placed between them; but the floods scoured out the piles, and at the end of the season many of them were found on the site where they had been driven with their points uppermost. At this work reliance was placed exclusively on trees laid flat, and weighted with stone in nets. These sank into the quicksand, but were not swept away. As they sank, others were added, until they formed a breakwater about 50 feet deep and 150 feet wide; and the bed of the river rose on the down-stream side until it became level with the adjoining land. The ground thus formed was planted as it became available, and the main stream was gradually deflected from its dangerous proximity to the northern shore, towards the centre of the bed.

The fourth main work, Q R, adjoining the northern abutment of the bridge, completed what remained unaccomplished. The deep channel moved away from the bank, and the river bed in which the two first spans occur grew up to ordinary flood-level.

The crumbling banks of the river immediately above the bridge being protected by the training works, and the shores commencing to gain on the river by the silting process, it seemed likely that the Chenab would, in the Wuzerabad reach, revert to its proportions as they existed before the construction of the trunk road embankment. To insure this, however, it was essential that the central island should be removed. The current set against the island as was expected, but the latter, being composed of somewhat tough soil, overgrown with jungle, did not disappear so rapidly as was desirable. To expedite the process, three cuts of varying dimensions were made, to ascertain whether the admission of a current would not widen the cuts by undermining their banks, and so hasten the demolition of the island; but the result proved that unless a large expenditure were incurred for channels of considerable width, the desired effect did not follow, because the Chenab contained too great a body of water to be influenced by channels very long in proportion to their width, and they were consequently abandoned. The current, however, continued to sweep away the central island from outside, and it also carried off most of the island L, which degenerated into a sandbank covered by ordinary floods.

On the adjoining Punjab and Delhi line, which was let by contract to the firm of Brassey and Co., the use of Kennard's sand pumps for sinking wells was introduced for the first time in India. A cylinder having a movable bottom, provided with an upright pipe extending to within a few inches of the top, in which air valves were fixed, was surmounted by a smaller cylinder fitted with a piston. The pump being lowered into the well, the chamber was exhausted by the piston worked by men from above, and the sand consequently flowed up the pipe and fell over into the larger cylinder. When full, the apparatus was hoisted and placed on a trolly; the bottom was detached with the cylindrical mass of sand upon it, and wheeled away. Another bottom was fitted to the pump, which was then lowered into the well and the operation repeated. This was a vast improvement on the native jahm previously in general use; but it was a clumsy apparatus after all. Large pumps required a steam hoist, and these alone produced much impression on the sinking. Smaller pumps, raised and lowered by manual labour, made but slow progress. The pumps, shear legs, steam hoists, and tackle were expensive in the first instance, repairs were frequent and troublesome, and the constant shifting of heavy plant for each length of well under construction consumed much time at critical periods of the season. Nevertheless, a great deal of good work was turned out by sand pumps, and Messrs. Brassey and Co. used them exclusively for well-sinking on the Punjab and Delhi railway. It was proposed by Government to adopt, on the Punjab Northern State railway, the system of bridging carried out on the Delhi and Lahore line. The large bridges were to consist of single well piers, 12 feet 6 inches in external diameter, sunk 40 feet, and carrying lattice girders under the rails, 97 feet 6 inches from centre to centre of the piers. Messrs. Brassey and Co's. plant was to be taken over, and the works were to be executed in the same way as at the recently completed Beas and Sutlej bridges. Meanwhile houses were built for the staff, as also workshops, stores, brick and lime kilns, materials were collected, tramways laid down, a colony of workpeople were established, and arrangements made for providing them with necessary supplies, between January 1870 and September 1871, in addition to carrying out the training works to prepare the bridge site. The works were carried out from the commencement by the same staff, although several changes took place among the chief engineers in charge of the railway. Mr. Alexander Grant held the office during more than half the time the bridge was under construction, and was present at the close of the operations.

Observatory towers, about 50 feet high, were built on either shore, 2 miles apart, each tower being provided with a fixed transit instrument by which the bridge could be ranged throughout.

The staff were established at Wuzerabad and Kathala, the principal workshops and the head-quarters of the undertaking being located at the latter place.

The new mètre gauge was much discussed, with reference to the Punjab Northern line, which was an extension of the existing railway system to the north-west frontier. After much argument the advocates of the mètre gauge prevailed with the Government as respects the Punjab Northern equally with other State railways, and the girders for the bridges were ordered for mètre-gauge weights only.

During the rainy season of 1871, the highest floods within the present generation occurred in the Punjab rivers. The river became a roaring torrent, covered with tumbling waves, and bearing along the wrecks of houses, trees, and dead cattle; but the Chenab training works were uninjured, though still in an unfinished condition, but on the Sutlej and Beas rivers, the railway bridges were partially destroyed, by the sand being scoured out to the foundations of the wells, which toppled over, falling up-stream. These accidents led to a total change in the designs of the Punjab Northern railway bridges. The Government determined on reverting to Mr. Power's three-well system, and, in the case of the Chenab, to sink the wells 70 feet. The Consulting Engineer for State Railways in India designed the superstructures of the piers and abutments. The former were 35 feet long and 8 feet 8 inches thick, with semicircular ends, standing on similar basements 38 feet long, supported on diminishing arches between the wells. These were to be sunk 6 inches apart. The abutments were each to be on a cluster of fifteen wells, sunk to the same depth as for the piers, and in two rows parallel to the central line.

This change increased the quantity of brickwork originally proposed five or six times. The Sutlej bridge was about 6,000 feet in length, on single well piers sunk 40 feet. It was proposed that the Chenab bridge should be about 9,000 feet in length, on three well piers sunk 70 feet. The former bridge, though a large work in itself, thus became a small one by comparison. Messrs. Brassey and Co. built it within four years, and it was proposed to build the Chenab bridge in the same time, under the Government departmental system.

At the close of the rainy season of 1871, the curbs for several

piers were pitched in favourable positions directly the river fell below the level of the higher sandbanks.

It was now determined to adopt Warren girders, and to have the rail level at the bottom; also to have sixty-four spans, 142 feet from centre to centre. The bridge was thus commenced on the 1st of November, 1871.

The bridge over the Pulkoo back channel was arranged to consist of nine spans, 43 feet 6 inches from centre to centre, with the piers of single well cylinders, as originally proposed for the main bridge, sunk into the clay substratum underlying the river bed. In the case of the back channel this was found to be about 5 feet higher than in the main stream. The girders were of plate iron of the ordinary pattern, carrying the rails on the top. This bridge was larger than necessary, now that the Wuzerabad channel had been closed by the training works; but it was thought prudent to allow a considerable margin in the event of accident to the embankments. The effect of the embankments was to lower the flood-level in the Pulkoo 4 feet; consequently sufficient headway was obtained to obviate the necessity of rising gradients in the approaches. The abutments of the back channel bridge were constructed on a cluster of eight wells each, sunk to the same depth as the piers. Thus the total number of wells to be sunk in the Chenab was:—

Main bridge, 63 piers of 3 wells	189
2 abutments of 15 wells	30
Back channel, 8 piers of 1 well	8
2 abutments of 8 wells	16
	243

It was proposed to make the large bridges "level crossing bridges," to lay down a floor of asphalt on buckled plates at the rail-level, and to use the bridges for ordinary cart and cattle traffic in the intervals of the passage of railway trains. On account of the great length of the Chenab bridge, it was considered expedient to have two passing places where the structure should be widened for two spans at each place, to allow of road traffic coming in opposite directions. To effect this, the four spans in question were made proportionally stronger, and with longer cross girders. This also necessitated lengthening six piers by having four wells instead of three to each. Thus six wells were added to the work, making in all two hundred and forty-nine.

For setting out the bridge, standard rods were made in England, one set being delivered to the contractors for the ironwork, Messrs.

Westwood, Baillie, and Co., of London, the other being used on the bridge in India to check the working rods actually employed in measurement. A fixed number of the latter extended from centre to centre of the piers, formed of seasoned pine 3 inches square. Each rod had brass caps at the ends, carefully planed and adjusted, so that they butted close together when laid on piles previously set out from the observatories. In crossing the channels, long piles were driven from barges with a monkey and steam hoist, and on these, sill timbers were bolted and made level and straight, thus enabling the measuring rods to be correctly laid. The distances were also checked by instrumental triangulation. At each pier site, a pile was driven, and the exact centre indicated by a nail. Corresponding piles at right angles up and down stream determined the axis of the pier, and the curbs were pitched accordingly. This was the process adopted on ordinary sandbanks left dry at low water; but in crossing channels, it became necessary to construct artificial islands, from 40 to 50 feet wide, and 80 to 100 feet long at the top, to provide space not only for the pier, but for building materials, engines, and the rails used as weights in well-sinking. The islands were formed by driving piles, about 3 feet apart, around the top boundary, and at the up-stream side in cut-water shape. At the other sides where there was no risk of sand bags being washed through, and under the curbs, the piles were farther apart. Gunny bags, filled with sand and sewn up, were brought out in boats, and laid down outside the piles, native divers being employed to place them in regular position under water, until they appeared above the surface, when a regular slope was formed and carried up beyond the level of winter floods. Loose sand brought out in boats was then put between the casing of gunny bags until it reached a similar height and formed the island. On this the measurement for the pier was made, and the curbs pitched accordingly. The wells were then built up for the first length and sunk through the island, and so on until the full depth had been attained.

During the last quarter of the year, or the first half of each working season, the river was always at its safest. The water-level subsided gradually until the middle of December. From the middle to the end of December, the lowest water-level of the season generally prevailed. Then came the winter rains, causing moderate floods, and the river fluctuated until April, when the snow began to melt, and the amount of water steadily increased. This continued up to the annual monsoon. Hence reliance was placed on the well-sinking, which could be executed from October

to December inclusive. By having everything fully organised at the close of the rains, and beginning first on the wells dangerously situated, it was in most cases possible to sink the first two lengths before there was a chance of their being overturned by a winter flood.

The curbs were of hard timber, built up of wedge-shaped segments below and additional flat covering pieces above, all breaking joint, and forming a ring of 2 feet broad and 2 feet deep. The component parts were securely fastened together with $\frac{3}{4}$-inch bolts, and the bottom of the wedge was armed with a cutter plate of $\frac{1}{2}$-inch iron 12 inches deep, riveted to a ring of 3-inch angle iron, which was again bolted to the timber, the wedge pieces or segments resting thereon. The outside diameter of the curb was 12 feet 6 inches, corresponding to the size of the well. The steining of the latter was 3 feet 3 inches thick, reducing the internal diameter to 6 feet; therefore the brickwork was gradually corbelled inwards from the curb until the full thickness was obtained. Eight tie-rods were placed at equal distances around the curb, and were continuous throughout the steining. The wells were built in five lengths of 14 feet each. At the top of each length, a ring of bar iron was laid in the brickwork, the tie rods passing through it and being screwed down tight by a nut 6 inches in length. The next length of tie-rod was screwed into the upper half of this nut or coupling, and so on. Thus each length of well was a compact cylinder in itself, capable of being pulled from side to side while sinking without injury to the brickwork. The curbs, which weighed 3 tons each, were put together in the workshops, and were moved out on the tramways running parallel to the bridge. From the tramway wagons they were slid down to their sites on inclined planes formed of rails.

It was found necessary to have a tramway both on the up and down stream sides of the bridge. These, with numerous connections and sidings, were in constant use by night and day during each working season. About 15 miles of tramway connected the various brickfields, workshops, sawing and mortar mills, &c., with the bridge and with each other. Piled bridges, to carry the tramways, were built of sufficient elevation to allow winter floods to pass; they were removed at the end of each working season, and re-erected at the commencement of the following one. During the busiest part of each year these means of transport were insufficient, and hundreds of camels, bullocks, and horses were employed as a supplementary measure.

The machine tools were not numerous, considering the magnitude

of the undertaking; nevertheless they were procured with great difficulty. In the principal workshop there were a couple of lathes, a couple of punching and shearing, one drilling, one planing, one plate-bending, and three screwing machines. Also a couple of blast fans for smithy and foundry. In the saw mill there were four circular saws and one vertical saw, one pendulum saw, and two sharpeners, besides about thirty engines of sorts, and a dozen mortar mills. These were all that needed steam. The usual small tools also were not procured without much difficulty and delay. The workshops were provided with a small foundry in which all castings needed for repairs were made.

The brickwork in the wells was composed of radiated bricks, headers, stretchers, and bonders, laid in hydraulic mortar, composed one-half of lime and one-half of ground bricks or soorkee. The lime was one-third stone and two-thirds block kunkur, the former being brought from Darapore, north of the Jhelum, and the latter from the vicinity of Sealkote. It was found by trial that the kunkur should be broken into pieces about 1 inch cube, then mixed with charcoal in the proportion of 1 part of charcoal to 2 of kunkur, and burnt in high kilns. The kunkur was drawn hot, and ground unslaked in the steam mills, coming out of a greenish colour. It was then mixed with the stone lime and soorkee, and placed in another mill, where water was added, and the whole ground together until thoroughly amalgamated. The mortar was immediately loaded into wagons and run out into vats alongside the wells in progress. It set at once in water, and was of excellent quality. The manufacture was in charge of European subordinates, who gave their whole time to its supervision, and soon became expert in detecting discrepancies in quality as compared with the standard. The kunkur deteriorated unless ground while hot. Without the stone mixture it was a total failure. The stone lime by itself would not set in water, but was used for superstructures in the proportion of one-third of lime to two-thirds of soorkee. For upper work it made excellent mortar. It was obtained by burning the boulders found in the beds of nullahs running into the Jhelum opposite Chillianwalla. All the lime used in the bridge had to be carried a long distance either on camels or by boats, but the trouble and expense were amply compensated by the result.

When each set of three curbs had been correctly pitched, they were sunk their own depth into the sand to steady them, and were carefully levelled. The tie-rods, with ring at top, were then fixed, nd building commenced. This was carried on until the ring was

reached, when the nuts were screwed down tight and the loading was put on.

One of the difficulties which led to serious complications and expense was the insufficient supply of rails for weighting the wells. Without heavy loads they could not be sunk during a single working season, nor could they be sunk vertically, or at a reasonable cost. The necessary quantity increased with each length of well from 25 to 300 tons, the latter quantity being needed to make the curbs enter the clay substratum. As the wells were so close together, no weights other than rails could be packed into the available space, each well having necessarily to be quite independent of its neighbours. The rails were laid in the first instance on strong beams resting on the brickwork, and placed in two rows, one on either side, leaving the 6 feet internal diameter clear for drawing up the sand from below. A packing of deal battens was then laid from one row to the other. On this another pair of tiers was laid, and so on until the weights proportionate to the particular length of well being sunk had been arrived at. On the top was erected the shear legs for hoisting the sand excavated. Owing to the number and depth of the wells, and the insufficient supply of rails, the latter were constantly being shifted from one well to another. When each length was sunk, the shear legs and rails had to be unloaded, that the next set of rods might be fitted and the next length of brickwork built up. Where the piers were being constructed on artificial islands, and when there were not sufficient rails available, the latter had to be transferred by boats between the islands backwards and forwards. This caused such delay that well-sinking had to be continued until after it had ceased to be safe. The whole mass of rails, engines, materials, and plant of various kinds used in the construction had to be afterwards removed to the shore for safety during the flood season; and always in the face of the rapidly rising river, and concurrently with a sudden desertion of men.

There are not many more inhabitants in the neighbourhood than suffice for its ordinary wants, hence much labour had to be brought from distant places. The crowds of strong men who did most of the heavy work during each winter were Afghans and other people from the hill country; and nothing would induce them to remain in the plains when once the summer had set in. Nevertheless, by working night and day, in no case was a well left unfinished before a flood season, nor were any tools or plant lost.

During the first working season, sand pumps were largely used for well sinking; but they were soon superseded by Bull's dredgers.

It was found expedient to have only two sizes, viz., a hand dredger with a capacity of 2 cubic feet, and a steam dredger of $7\frac{1}{2}$ cubic feet. This latter was the largest size that could be conveniently worked in a 6-feet well, and with the steam hoists provided. Three steam hoists were placed in a flat-bottomed barge, 60 feet in length and 12 feet in width, moored alongside the artificial island on which the pier was in progress. The barges were warped across the stream from island to island, and were used for loading and unloading rails in addition to well-sinking. In jerking the large dredgers it was found necessary to use a lever on a tripod stand, furnished with a sling and claw to grasp the chain; also rocking shear legs, by which the dredgers discharged themselves over the side, being slung from an auxiliary chain during the operation, in order to liberate the arms of the machine and permit it to open. The dredger was only suited to excavate sand; and as several intermediate strata of clay were met with, as well as boulders and sunken trees, other means had to be adopted for effecting the excavation. Skeleton dredgers were made, following the same principle as the bucket dredgers, but consisting entirely of ribs, armed with steel points. These sufficed to break up ordinary intermediate clay strata sufficiently to allow of the material being subsequently dealt with by the bucket dredgers. When a deep hole had thus been excavated in the centre of the well, by piling on weights above, the wedge-shaped curb generally forced the clay from under it into the hole, and the well descended. The operation was repeated until the stratum was pierced. On coming to obstacles in sinking, the water was removed by a "mote," or leather bag of the country. Three of these, of large size, worked by eighty men each, were usually sufficient to reduce the water, so as to set the sand in motion, as also to get some benefit from the weight of the brickwork, and so make the well go down. Under these circumstances a "blow" often took place from outside, when a quantity of sand rushed up from below, and sometimes nearly filled the well. If these expedients did not move the well, helmet divers were sent down to clear away the obstruction.

When a well left the perpendicular in sinking, it could usually be straightened up to the third length, or when it was about halfway down. In such instances, the next length was carried up in the plane of the crooked well, and the whole was pulled over when sinking was recommenced. This was done by dredging outside the well on the elevated side, increasing the weight of rails on that side, and using small dredgers only, pulled over so as to excavate close to the curb. Strong cables with powerful blocks and crabs

kept a strain from the same direction, while timber struts on the depressed side helped to push the well back to the vertical, when it began to sink. If the well was not vertical after it was half-way down, it was hopeless to attempt improvement, and the rest of the brickwork was carried up vertical in each length. Boulders, lumps of hard clay, and sunken trees frequently tilted the wells out of the perpendicular, and had to be cut out by divers before the sinking could proceed. Four plumb-bobs were hung outside each well, as a guide to the overseers in charge. On reaching the clay substratum, the greatest weight was needed to make the curb enter. This was rarely accomplished without many "blows," and much repeated clearing out of sand and water. When the surface of the clay was level, and there were no faults, the water could be entirely exhausted by the motes. Labourers then descended and excavated the clay until the well sunk to the desired depth. The water frequently burst in upon them under so great a pressure as 70 feet, and filled the well, compelling a speedy retreat. When the surface of the clay was inclined, or when faults were present, the wells could not be freed from water with the means provided, and it was necessary to maintain a prolonged series of dredging and diving operations to found the curbs securely in the clay.

After the wells were finished, they were filled with hydraulic concrete, composed of mortar (as for the brickwork), and ballast consisting of bricks broken to 2-inch cubes, the mortar, when dry, being one-third of the total bulk. The puddling was done on sleeper platforms adjoining the wells in course of being filled. When the wells were free from water, the concrete was thrown from the top; otherwise it was lowered by skips. In but few cases was it possible to build the pier basements in the same season in which the wells were sunk, because they had to be founded at low-water level, and the floods had generally commenced before the wells were down. Where circumstances permitted, a row of sheet piling, backed by fascines, to check the running sand, was placed around the pier, and the water removed by a centrifugal pump and portable engine, to allow of the basement arches being turned on the tops of the wells. Otherwise it was necessary to turn all the arches of the season during the short time that the river remained at low-water level.

As it was necessary to carry on the well-sinking after the snow floods had begun, the brickwork was raised several feet above low water, to keep the stacks of rails out of reach of the current, and to admit of the water being drawn out for sinking into the clay substratum, and for filling in the concrete when possible. These

extra lengths were pulled down during the following season. The upper stratum, 2 feet deep, of the concrete was excavated and filled in with solid brickwork. From the wells thus levelled off at low-water mark, the arches were sprung, carrying the basement. By the advice of the Consulting Engineer to Government, a belt of plate iron, 12 inches deep by ½ inch thick, was fitted around the tops of each set of wells to keep them from falling away from each other.

The greatest number of wells sunk in one season, out of the three in which they were completed, was one hundred. The greatest number of bricks used in a day was 1 lakh.

Considerable difficulty was experienced during the first season in sinking the wells so close together as 6 inches, and several ran foul of one another. During the second season the distance was altered from 6 inches to 2 feet, and this answered all requirements. In the third season the distance was reduced to 18 inches. The outside wells of each pier were kept half a diameter in advance of the central one, to counteract the tendency to draw towards the centre. When the wells were ready for the basement the measurement of the spans was repeated, to provide for lateral displacement in sinking. This amounted, on the average, to 3 inches throughout the bridge.

The clusters of wells supporting the abutments were much more troublesome than those of the piers, owing to the two parallel rows having been designed only 2 feet apart. The wells composing each row did not materially interfere with each other; but whether sunk alternately or simultaneously, one row ran into the other, owing to the large area of sand disturbed in the same locality. This caused much delay; but eventually all were sunk without injury. The following is an abstract of the average progress made in well construction throughout the works:—

Occupied in construction	188 days.
Actual number of working days	123 „
Lost in waiting for rails	28 „
Lost through winter floods, native holidays, rain, &c. . .	37 „
Maximum depth sunk per day of twenty hours (two shifts) .	5·48 feet.
Sand sinking „ „ „ „	2·67 „
Clay „ „ „ „ „	0·53 „
Maximum sinking ever attained in an isolated case per day of twenty hours	9·85 „

The superstructures of the piers and abutments were built 20 feet in height, and finished with a cornice. Bed stones extending from face to face, 4 feet 4 inches wide and 1 foot 3 inches thick, were built into the brickwork, and in these were set the lewis bolts of the cast-iron knuckle plates supporting the saddles which were bolted to the girders. The wing walls were coped

with stone and finished with large blocks at either end. The material was sandstone, quarried in the Salt range, and carted from a distance of 60 miles.

With a view of assisting the piers to resist side currents, provision was made for the expansion of the girders in every second span, so that the piers were tied together in pairs by fixed bearings. In the alternate rolling spans the expansion was 1½ inch, and in the fixed spans ¼ inch. The latter was obtained by the saddles overriding the fixed knuckle plates, the difference being taken up in the girders.

Towards the end of the first season, a loose filling of 30,000 cubic feet of boulders was ordered to be laid around each pier, to sink into the holes formed by scour, and to act as a protection. Subsequently it was resolved to place a similar filling of 400,000 cubic feet of boulders around each abutment and under the two adjoining spans, with a row of rectangular blocks or wells from the abutment to the second pier. Arrangements were accordingly made to collect all the boats procurable on the Chenab and adjoining rivers, of such size as were suited to the precarious navigation between the bridge site and the rapids.

The difficulties in getting stone were now greatly increased by the enhanced demand for pier protection. Nothing could be done before the floods had subsided, and the second working season had commenced. As supplies came in but slowly, recourse was had to concrete blocks. These varied in size from 3 cubic feet to 27 cubic feet each, and were piled with the boulders around each set of pier wells as the latter were finished; but owing to the short supply of rails for weighting the wells, many were not sunk until just before the river became unmanageable, and but little protective filling could be placed around the piers. Attempts were made to reach them by boats afterwards, but several wrecks occurred, and the conveyance by boating proved more expensive than the whole sanctioned rate for the material, inclusive of manufacture in the case of concrete blocks, or collection and transport from Cashmere territory in the case of stone. Thus it was found useless to attempt laying down any blocks after the tramway bridges had been removed at the beginning of each flood season, the piers being then inaccessible to boats. When the main stream attacked any of the piers, 30,000 cubic feet of stone or concrete soon disappeared, and the quantity had to be increased to 40,000 cubic feet. In some cases 100,000 cubic feet were laid down.

The bridge was carried on from both ends; the organisation being mutually independent, and under separate staffs of engineers.

It was hoped that in the third season the main channel of the river would move northwards, and pass through the finished part of the bridge, thus enabling the work to be completed on a sandbank dry at low water. To aid this some spurs were thrown out, and channels dug as far as available funds permitted, but the operations proved expensive and were abandoned. The wells were, however, sunk across the minor channels at the end of the second season, and from both shores up to the limits of the main channel, viz., about three-fourths of the bridge. The main channel then occupied the course shown by the black dotted lines on the plan.

At the beginning of the third season, the main channel was nearly unchanged. The temporary bridges employed in carrying the tramways were constantly swept away. The artificial islands sank into the quicksand until they became of great dimensions from the additions made at the top. The waterway being thus contracted, the sand at the bottom of the channel was in motion. The current became very deep and rapid, and nearly all the water from the minor channels poured into the hole thus scoured. Apprehensions existed lest some of the islands should be undermined, and the wells toppled over before they had been sunk within safe limits. The works proceeded by night and by day in two shifts of ten hours each, the sands being lit up by numerous torches and large fires; and the engines were kept constantly running. Some of the boats were withdrawn from the transport of stone to take the place of the pile bridges scoured out, and several were wrecked. All the engines and appliances, and all the men were concentrated on this one point, and at last the bridge was closed in as regards well-sinking, but with the loss of several lives. The last well was sunk just at the beginning of the floods. Then the whole of the tools, plant, and stores had to be landed by boats, the piles and floating bridges having been carried away.

During this season, several girders were erected on piers situated on a large natural island which had been in existence during the progress of the works. So safe was this ground considered, that hitherto it had been the custom to store materials thereon during the flood season. Each complete span weighed about 90 tons, and the main girders had a camber of $2\frac{1}{2}$ inches. Their effective depth was $\frac{1}{15}$ span and they had no top bracing. Some were riveted on the sandbanks alongside the piers, and then lifted by a pair of Wellington cranes running on a piled staging. Others were built in place on timber stagings. No piece weighed more than 2 tons, so that ordinary carts sufficed for transport. In the case of one of the first set of spans erected on the island, the girders had been

riveted on piles driven 20 feet into the soil, and were ready for lifting, when a sudden flood occurred. The island disappeared, and the pair of girders with it. There was no resource but to wait until the river again fell to low-water level during the following winter, and girder erection had to be discontinued as no longer safe. During the ensuing winter it was found that the sand had closed in over the sunken girders, but a cofferdam was made around them; the water was pumped out, and the sand excavated, thus enabling the joint rivets to be cut out and the girders recovered uninjured. In the pumping operations Woodford's centrifugal pump worked in running sand when all other pumps became choked.

The long-pending discussion relative to the gauge question in India, particularly with respect to the Punjab Northern State railway, now finally resulted in a return to the standard Indian gauge of 5 feet 6 inches.

The question then arose as to how standard-gauge loads were to run over girders calculated for mètre-gauge loads. The former were taken at 6 tons on the wheel, and the latter at 3 tons, or ¾ ton per lineal foot. The girders had been designed by Mr. Rendel, M. Inst. C.E., Consulting Engineer to the Home Government, in accordance with instructions furnished to him, and were constructed by Messrs. Westwood, Baillie, and Co.

With a view to make the bridge suitable for greater rolling loads, it was thought expedient to abolish the buckled plates and asphalt for the Trunk Road traffic, and to lay the rails only on the cross girders, together with planked footpaths for platelayers. The saving in dead weight thus effected made the bridge available for a wheel load of 4 tons, the approved strain of 4 tons and 5 tons per square inch in compression and in tension, being maintained under the test load adopted of two engines and tenders, with the rest of the span covered with loaded wagons. The main girders were also set 1 foot farther apart, to admit of standard-gauge rolling stock passing freely, and the cross girders were lengthened accordingly. This made the bridge suitable for light engines on the standard gauge, but whether it is to be strengthened for the ordinary engines, or whether higher strains are to be adopted to suit them, has not yet been decided.

During the flood season of 1873–74, a serious accident occurred to the Alexandra bridge. A great mass of protective material had been placed around the finished piers, presenting the appearance of a weir across the river; but, owing to the want of sufficient weights and other appliances, there had not been time to lay down

any around the newest wells before all had become submerged by the rapidly rising river. Under these circumstances the main stream struck the top of the central island at right angles, which thus became concave, instead of being convex as theretofore. The current consequently was diverted to the northern shore below the influence of the work M N. Here it met the remains of the island marked L, and was diverted back again towards the southern shore. The long row of blocks piled around the piers now made the current run parallel with the bridge. The whole force of the flood thus poured into the hole already existing at the site of the new wells. The bed of the river became disturbed, and the obstacles caused by the finished pier fillings created a scour and deep hole in front of each. As the current flowed at right angles to the normal course of the river, these holes soon ran into each other, until a deep trench was formed along the face of the bridge for more than a mile in length. The force of the current drove the bed before it, scouring out the sand. The impetus of the current carried the channel beyond the deep hole above referred to, when it suddenly turned at right angles through the bridge, and took its course towards the lower island. On the river falling low enough to make it safe to venture near, it was found that the up-stream and centre wells of three piers had disappeared, leaving the down-stream wells standing. The plate-iron belts had not answered their intended purpose; they had been broken; and the support of the down-stream wells had been of no avail. When the flood season was over, the position of the fallen wells was found by boring. The sand had closed in over them, and the river now showed a tendency to silt up on the spot where they lay. They were nearly parallel to the centre line of the bridge, in the direction of the current, and had evidently toppled over as a tall chimney is blown down in a storm. If undermined, they would have fallen up stream. The accident occurred merely from insufficient base, the depth of the wells being 70 feet, with a diameter of only 12 feet 6 inches.

As the sites of three piers were blocked so that fresh wells could not be sunk in the same place, and as all had fallen nearly parallel to the centre line, it was determined to shorten the first span by 20 feet, and to lengthen the last of the four spans affected, keeping the two middle ones of the normal dimensions. This was effected by abandoning the standing down-stream wells in each of the three damaged piers, and by sinking new triplets on the north side of each. The girders of the first span were cut to suit the shortened span, and new ones were ordered from England

for the lengthened span, which preserved the same general appearance as to depth, &c., the extra strength due to the increased span being made up by stronger iron. The new wells were sunk during 1874–75, through artificial islands, and were finished in time to admit of 30,000 cubic feet of filling being placed around each of the triplets before the floods set in.

It was calculated that, at the end of this season, the girders would be erected for all the spans except fifteen, including the four for which the wells were then in progress, and as the main channel still kept its dangerous sidelong course, it was determined to divert it by attacking the central island. The operation proved successful, the current flowed straight through the bridge, carrying much of the island with it, while the sidelong channel silted up, thus enabling the fifteen spans of ironwork to be rapidly and cheaply erected on a sandbank dry at low water, instead of on an expensive staging in deep water.

The plan adopted was to round off the concave corner of the island, the material being run into a spoil bank placed to turn the stream, and shown in oval form on the plan. This led to the current flowing against the convex side of the island, and along its edge, which was purposely made vertical. A wide canal was also excavated through the island, and the material removed by ballast trains to form the new standard-gauge embankment across the low land between the Pulkoo and Chenab bridges. The meeting of the currents caused a deposit of silt between the lower end of the island and the bridge, and so raised the bed of the channel above low-water level, while the main stream not having time, in face of this obstacle, to turn round so sharp an angle as that presented at the lower end of the island, flowed straight on, thus cutting a new course near the centre of the bridge, under the girders already erected, in which position it met the piers in the direction of their length. The main channel now took the direction shown by the lines of black crosses, and the island was shorn of a large portion of its bulk, while the current still continued to cut into it by sidelong action.

At the beginning of the working season 1875–76, everything was completed except the superstructures of the three new piers, and the erection of fifteen spans, including the long one. The work proceeded rapidly on the sandbank thrown up by the river in consequence of the operations on the central island. The first train (mètre gauge) passed over the bridge on the 23rd of December, 1875, and ran to Goojrat on the new line just constructed. It was thus proved possible to carry out the greatest work in India

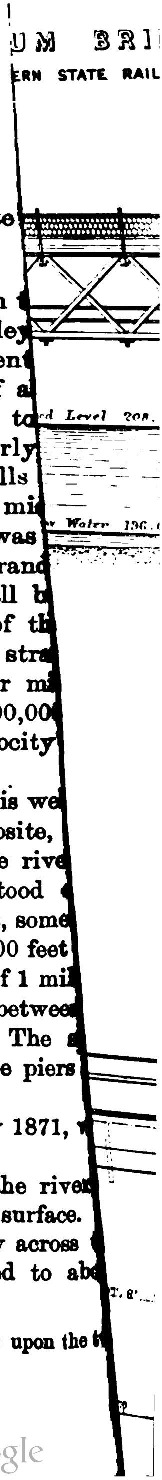
under the new State railway departmental system, in a manner which, as regards quality, cost, and rapidity of execution, would compare not unfavourably with the greatest similar works undertaken either by the officers of the principal guaranteed railway company, or by the principal firm of English contractors employed by an Indian company.

The cost of the bridge has been as follows:—

Sub-heads.	Rate.		Quantity.	Amount.	Total.
	Rupees.			Rupees.	Rupees.
Well curbs	1,237	Nos.	201	248,637	..
" "	837	"	24	20,088	..
Block "	136	"	96	13,056	..
Brickwork	35	100 c. ft.	1,922,550	672,892	..
Well and block sinking . .	26	..	2,048,218	532,537	..
Dredging for placing filling around piers and abutments	100	Nos.	65	6,500	..
Concreting wells and blocks .	25	100 c. ft.	511,569	127,892	..
Pier belting	350	Ton	112⅓	39,317	..
Boulder and block filling. .	20	100 c. ft.	2,570,000	514,000	..
Wrought-iron girders. . .	28,760	Span	63	1,811,880	..
Ashlar	5	Cub. feet	6,009	30,045	..
Converting spans from mètre to light standard gauge .	800	Span	63	50,400	..
New lengthened span, No. 49	50,069	"	1	50,069	..
Shortening No. 46 span . .	200	"	1	200	..
Extra well curbs for three new piers	1,237	Nos.	9	11,133	..
Extra brickwork " "	37	100 c. ft.	59,535	22,028	..
" sinking " "	51	"	79,799	40,700	..
" concreting " "	25	"	18,383	4,596	..
" pier belts " "	350	Ton	3	1,050	..
Additional boulder filling, fifty-nine piers at 10,000 cubic feet, each	20	100 c. ft.	590,000	118,000	..
				4,315,020	..
Contingencies, at 5 per cent..	..	..	..	215,751	..
				4,530,771	
Abstract of back channel bridge	..	..	..	181,409	..
" embankments connecting bridges	..	..	..	46,837	..
Abstract of river training works	..	..	..	612,879	..
				5,371,896	5,371,896
Permanent charges for tools and plant at 5 per cent. on works estimate	..	..	..	..	268,594
Staff quarters, permanent charges	..	..	..	..	17,884
Local establishment charges at 15 per cent. on works estimate	..	..	..	..	805,782
					6,464,156

The total cost of crossing the Chenab may therefore be taken at about 65 lakhs of rupees. This is irrespective of proportional charges for establishment at headquarters, and the cost of permanent river conservancy, due to the bridge site, as also of any strengthening of the girders, should such be decided on.

The Paper is illustrated by several diagrams, from which Plate 4 has been prepared.



No. 1,528.—" The Jhelum Bridge, Punjab Northern State
By Frederick Morris Avern, M. Inst. C.E.[1]

The Jhelum, anciently known as the Hydaspes, rises in east of Cashmere, and flows tranquilly through the valle by Srenuggar, the capital of the country. It subsequen the Baramuta Gorge, and after a tumultuous course of a miles among the Himalayas, descends by easy rapids t above the town of Jhelum, where it becomes an orderl and pursues a course of 200 miles farther, until it falls Chenab. At the town of Jhelum, a little beyond its mi the bed has a width of about 5,000 feet; and this spot was for the railway crossing, as being on the line of the Gran road, the traffic of which is carried over the river in all seasons by a bridge of boats. At this place the bed of t is of sand, 15 to 20 feet deep, overlying a thick str boulders and shingle; the fall of the bed is 1 foot per m the approximate discharge of the river at high flood 200,00 feet per second; the maximum recorded surface velocity 8·66 feet per second.

The right bank of the river at the town of Jhelum is w solidated, and not liable to erosion; the bank opposite, 1¼ mile off, is also well consolidated; under this the riv many years ago, and the town of Jhelum once stood Between these two banks the course of the river varies, som running in a contracted bed under the right bank, 2,000 feet in the year 1857, but generally spreading to a width of 1 mi

The length of the bridge was fixed at 4,875 feet betwe abutments, with training works on the left bank. The decided on were fifty of 90 feet each, giving forty-nine piers two abutments.

A trial well was started on the left bank in January 1871, the results shown in the table on page 95.

Boulders and coarse sand extended nearly across the rive the Jhelum bank, at which point they approached the surface. lower clay stratum also extended nearly continuously across river. In the middle of the river bed it thickened to ab

[1] The discussion upon this Paper was taken together with that upon the Papers preceding, and occupied portions of two evenings.

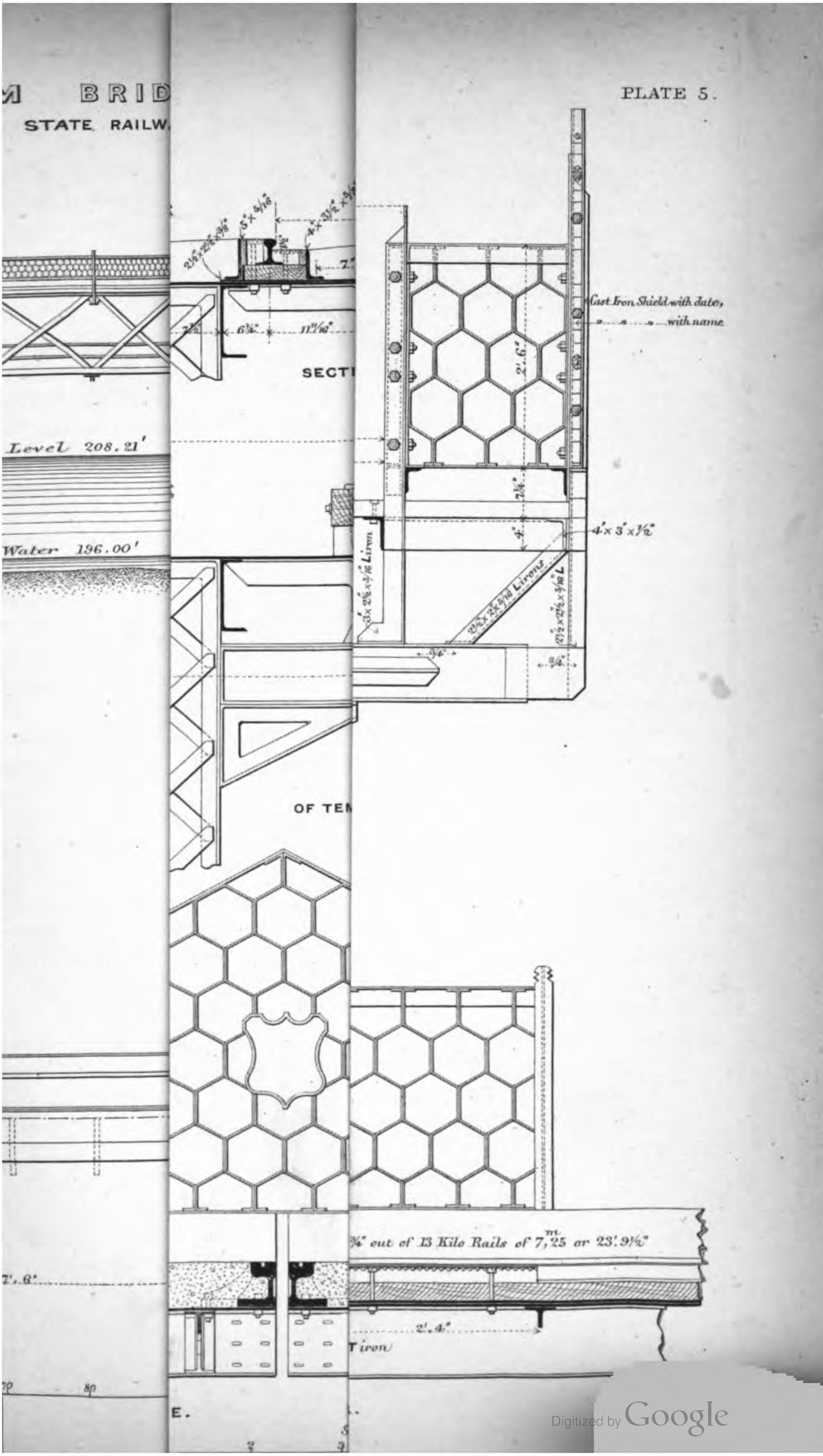


PLATE 5.
BRID
STATE RAILW
Level 208.21'
Water 196.00'
SECTI
OF TEM
Cast Iron Shield with date,
with name
2'.6"
4"x 3"x ½"
Lirons
L iron
out of 13 Kilo Rails of 7,25 or 23'.9½"
2'.4"
T iron
E.

15 feet, thinned again to 10 feet, and then thickened out at the Jhelum abutment to 13 feet.

The foundations of the right abutment and piers 1, 2, and 3 were formed in this stratum of clay; all the other piers were

Number of different Strata.	Depth of successive Strata, commencing from the Low-water Level.	Nature of Strata.
	Feet.	
1	23	Sand.
2	9	Coarse sand and boulders.
3	2	Coarse sand, boulders, and clay.
4	3¾	Coarse sand and boulders.
5	1	Clay.
6	1¾	Coarse sand and boulders.
7	2½	Coarse sand only.
8	6½	Fine sand.
	49½	Feet below low water.

founded in the boulder stratum, having been sunk down about 12 feet into it. The foundations were brick cylinders, three in a line transverse to the bridge, a centre one 12 feet 6 inches in external diameter, and two flanking ones, each 10 feet in diameter. Having regard to the bed of boulders in which they were founded, a substantial wrought-iron curb was adopted, on which to build the brickwork, and to which it was bonded by circular flat iron rings and vertical round iron tie-rods. Three rings were introduced in the 32-foot length of brickwork at intervals of 10 feet, and the tie-rods starting from the curb went through the whole length of brickwork in the well. In the smaller wells there were eight rods, and nine in the larger wells. The rings were of flat iron, 3 feet broad by $\frac{5}{16}$ inch thick, and the rods were 1 inch and 1¼ inch in diameter. The brickwork of the smaller wells was 2 feet thick; that of the larger wells was 3 feet 3 inches thick. It was composed of specially made radiating bricks, set in mortar consisting of 1 part of stone lime, 2 parts of kunkur, and 6 parts of fine brick-dust, the whole well ground together in steam mortar mills.

The first process of sinking the wells where the river bed was not dry was to form islands at the sites of the piers, whereon to pitch the curbs. These islands were formed in still water by putting out circles of sand bags round the pier sites, and filling the interior space with sand. Where there was a current of water,

as in the main channel under the right bank, the stream had to be diverted or the current slackened by tree spurs constructed up the river at the heads of channels. This operation was most successfully carried out, so that islands were even constructed in water usually 10 or 15 feet deep, flowing with a strong current.

The islands having been formed, the curbs were pitched with a clearance of 2 feet between them. The triangular space formed by the hollow of the curb in cross section was then filled up with concrete, and the vertical bolts having been fixed to the curb and connected on the top by an iron ring, the brickwork was started and carried up to a height of 20 feet. A staging was then formed on the top of the well, a winch set up, and the excavating tools set to work. Sand pumps and jhams were the first tools employed for getting out the sand; but as the work progressed, Bull's dredger was substituted.

The wells went down very easily through a stratum of 20 feet of sand (Plate 5, Fig. 2), only requiring to be carefully watched to keep them plumb. After this the difficulties of the work began. The shingle bed now reached was a formidable obstacle to further rapid progress. This bed consisted of stones varying in size from a pigeon's egg to 40 lbs. in weight; and occasionally the stones, instead of being mixed with sand only, were conglomerated into a solid stratum.

The only means of getting the wells through this was by the use of large and powerful sand pumps, fitted with bottom pipes 10 inches in diameter. The largest pumps were 2 feet 10 inches in diameter, 3 feet high in the cylinder, and having a top barrel 1 foot 1 inch in diameter, and 2 feet 2 inches high, fitted with a plunger $11\frac{3}{4}$ inches in diameter. This pump was provided on the underside with powerful prongs for loosening the boulders, by which, as the stones became loosened they were sucked through the pipe at the bottom of the pump into the cylinder, which when full was hauled up the well, and the contents emptied out on to a stage. This tool did a great part of the work; the rest was effected by divers with picks, the divers being always put into the wells when the progress with the pumps was not satisfactory. Later on a Woodford's pump was also found useful, as by pumping out the water from the interior of the well, the unbalanced exterior pressure blew in the material surrounding the curb, and often gave the well a fresh start after it had been sticking.

By these means, and by loading the top of the well with rails, weighing in the aggregate generally about 340 tons, the wells were satisfactorily put down to an average depth of 35 feet

below the sandy bed of the river, and 32 feet below the lowest water-level. For the last 15 feet the sinking was generally through a solid bed of shingle.

Only one mishap occurred during the work, at the end of the first season's sinking operations. The wells in the tenth pier from the left bank (No. 40) had been partially sunk, when a flood came down and overthrew all three wells. After extensive borings these wells were found almost clear of the site on which they had been pitched; and by a slight deflection in the position of one of the 10-foot wells, three new ones were got down at the proper site for the pier. A repetition of such an accident was guarded against by starting only in one season the number of wells that probably could be sunk before the time of heavy freshes; but when any remained uncompleted, their tops were cut off above low water. The wells when sunk to the full depth were filled with concrete, and connected by small arches turned over the 2-feet clearance space.

The superstructure of the piers (Fig. 3) was then commenced, 27 feet 6 inches long and 7 feet 6 inches wide, with semicircular cut-waters. They were carried up 21 feet above low water, the last 4 feet consisting of a bold brick Doric cornice, and a blocking course containing bed stones 1 foot 3 inches thick.

The abutments are each founded on three wells, 12 feet 6 inches in diameter (Fig. 4). They are similar to the piers, but are flanked with longitudinal wing walls. The south abutment wing walls are 68 feet 6 inches long, founded each on six 10-foot wells, and are tied together by three cross walls. The north abutment wing walls are 50 feet long, founded on the solid ground of the right bank, and tied together by cross walls. The spaces between the walls are filled with earth. The wing walls are carried up to the top of the girders, and surmounted by a stone parapet.

Well-sinking was commenced in October 1871, and completed in April 1874. The working season of each year was from October to April, with occasional interruptions from freshes. The whole operation of sinking forty-nine piers, two abutments, and the wing walls on the left bank, comprising one hundred and two wells 10 feet in external diameter, and sixty-three wells 12 feet in external diameter, thus occupied eighteen months, giving an average of nearly nine wells per month. In sinking these wells, 20,766 cubic yards of material had to be removed from the bed of the river at an average cost of 17*s.* per cubic yard. The superstructure of piers was commenced in November 1873, and completed in March 1875.

The total quantity of brickwork in the piers and abutments is 22,589 cubic yards, built at a cost of 18*s.* per cubic yard. The internal hearting of concrete within the wells amounted to 6,888 cubic yards, at a cost of 8*s.* per cubic yard. The cost of specially made radiating well bricks was 30*s.* per thousand, that of lime 35*s.* per 100 cubic feet. Pounded brick (used in place of coarse sand in mortar) cost 17*s.* 6*d.* per 100 cubic feet. Fuel cost 49*s.* per 100 maunds. Labour of building, mixing mortar, watering, carriage of materials, and other charges, amounted to 17*s.* 6*d.* per 100 cubic feet.

The protective works at the Jhelum are of a simple but permanent nature, and consist of only one groyne or spur of earth, 4,800 feet in length, 8 feet in height, with a width of 8 feet on the crest, and with slopes of 2½ to 1. At the toe of the river slope, a trench 10 feet wide is dug down to the lowest water-level, and then filled with boulder stones. On this base stones are piled up to the crest of the embankment, the face of the stones being dressed off to a slope of about 2 to 1. Should any scour occur in front of the spur, the boulders fall down and fill up the hole. The crest and rear slope of the spur are planted with willows, and the slope is turfed. The spur is in two lines forming a sharp angle at their intersection, and this is made into a very strong bluff formed by a great mass of stones backed with earth. In one season a powerful stream, having a depth of 15 to 20 feet, flowed under this part of the spur; but beyond the falling in of a few stones, no damage has occurred, and no repairs have been required. The structure is in fact a permanent one, and the cost of maintenance will, so far as experience goes, be very light. The cost of the spur has been £5,574, or £1 3*s.* per lineal foot. Boulders from 8 lbs. to 40 lbs. in weight were obtained from the bed of the river a few miles above the bridge, at a cost of from 3*s.* 6*d.* to 7*s.* per cubic foot. The spur extends from the left abutment of the bridge to the high ground forming the left bank of the river. Its purpose is to prevent the river flowing under this bank and getting in rear of the bridge abutment, and it has enabled the bridge to be made 1,400 feet shorter than would otherwise have been necessary. The cost of lengthening the bridge to this extent would have been about £36,750, or nearly seven times as much as the cost of the spur.

As the piers of the bridge were sunk 15 feet into a solid bed of boulders it might be thought that they were amply secure, but as a further measure of safety, they were surrounded by an encircling armour of boulders, originally thrown in all round the pier, with a width on the top of about 2 feet, and a slope formed naturally by

the stones. The piers were each intended to be protected by 10,000 cubic feet of stones, and took originally an average of 8,000 cubic feet of stones; but during three seasons' floods, the stones have sunk in the strong current, and as they sank others have been thrown in, so as never to let the armour fall below the level of the plinth. In this way there is now an average amount of 13,467 cubic feet of boulders round each of the piers, with a maximum of 55,167 cubic feet round No. 3, and a minimum of 6,113 cubic feet round No. 46.

After each season's floods, the position of the boulders around the piers was ascertained by probing. From this it appears that in the strong currents the stones slip down from 30 to 40 feet below the piers, and that the lateral motion of the stones is slight. The boulders disappear chiefly by sinking into the hole scoured out immediately around the pier, and this action is limited at the Jhelum by the natural boulder bed; and it seems probable that when the stones put around all the piers have had time to sink to this level, no further supply will be necessary. The boulders are put in mostly when the river is low. Those put in during floods require to be of very great size, or to be confined in rope crates. The stones sink deepest on the up-stream side of the piers.

The value of the boulders consists in their forming:—

1st. A local supply of material ready to drop into any holes scoured out around the piers.

2nd. In giving support to the piers to resist the overturning action of the stream.

The value of this first service cannot be too highly rated; and the second is also important, where, as in the case of the Jhelum and most of the large bridges in India, the piers are constructed on the European principle. In Indian rivers the stream frequently runs at right angles to the axis of their beds, or parallel with the bridge, within local limits, and the lesson clearly pointed by this action is, that piers should have equal bases in all directions, should in fact be circular; and most of those engineers who have gained experience of well foundations in these peculiar rivers are now agreed that one large circular well would be the best of all foundations, both as a matter of security, and on account of ease and rapidity in sinking.

The superstructure (Fig. 3) is composed of a pair of lattice girders, connected on the top by an upper roadway of cross girders and rail-bearers, covered by buckle and flat plates, and braced on their underside by T irons and diagonal bracing, and footway planks.

The upper roadway (Figs. 5 and 6) carries the railway and two footways for platelayers, &c., and was also designed for ordinary road traffic; but this intention being in abeyance, the intended asphalt filling over the plates has not been laid down. The lower roadway is intended for foot passengers and mules and ponies only.

The girders are each 97 feet 4½ inches long, 10 feet 2 inches in total depth, and 9 feet 4 inches between the centres of the booms. They are divided into ten triangular bays, each of 9 feet 4 inches, and two end lattice box struts. The top boom is composed of a pair of channel irons 10 inches deep, connected at every 4 feet 8 inches by T-iron distance pieces. The bottom boom is formed of two flat plates, connected at every junction of the diagonal by T-iron distance pieces, and intermediately by short diagonal ties. The diagonal struts consist of two T irons braced together with flat lattice bars. The diagonal ties are plain flat bars. The vertical struts at the end of the girders are T irons braced together with flat lattice bars; they have a flat plate riveted to them on the underside. Each vertical strut has also riveted to it on the outer side of the girder a triangular standard, intended to give lateral stability. The diagonals are connected with the top and bottom booms by rivets. The web of the T-iron struts was sheared off at the top and the bottom booms for a length of 1 foot 9 inches, to admit of a distance piece being introduced. The connection was then made by eight ⅞-inch rivets, four of which passed through the boom, tie, or struts, and distance piece, and the other four through the tie, or strut, and boom only. On arrival at the works many of the T irons in the braced diagonal struts were found to be cracked at the ends, along the line whence the web had been sheared away. As four rivet-holes occurred in this line, the fracture gave rise to doubts as to the quality of the iron, and was the cause of the stoppage of work for nearly a year. This idea of the bad quality of the iron was strengthened from the fact that, the ironwork being supplied by two makers, nearly all the cracks occurred in that supplied by one firm. By order of Government some of the struts were submitted to test strains, with surprisingly good results, the iron standing a mean tensile strain of 24·89 tons per square inch. It thus became apparent that the defects were not owing to bad material, but to shearing away of the web, and subsequent bad packing, and consequent damage from handling in the course of their transit from England. These evils must, in the Author's opinion, be attributed rather to the designer than to the manufacturer or to the transport agents, inasmuch as the

cutting away of the webs was not warranted, and caused a serious loss in the sectional area of the struts. The defects were satisfactorily repaired by the introduction at the joints of connection-plates, forming an intermediate member between the diagonals and the booms. Their adoption was a troublesome matter in practice, involving many alterations and much extra labour. In addition to testing the iron, the Government ordered a complete span to be erected and tested with twice the load for which the girders were originally calculated. Accordingly, one span was loaded with 180 tons of dead weight, being nearly 2 tons per lineal foot. The result was a deflection of $1\frac{1}{8}$ inch only, $\frac{1}{8}$ inch of which was permanent. This being considered satisfactory, orders were given for the prosecution of the works.

The top roadway is formed of rail-bearers, resting on small wrought-iron saddles riveted to the cross girders. These rail-bearers are 1 mètre from centre to centre, and are covered by buckle plates 3 feet 4 inches by 4 feet $10\frac{1}{2}$ inches. The space between the rail-bearers and channel bars of the top booms is covered by flat plates $\frac{1}{4}$ inch thick and 3 feet $2\frac{1}{2}$ inches wide, extending across the top boom, so as to form a top plate for the boom as well as a floor plate. The cross girders project outside the main girders, and carry a standard for a hand railing. The space between the hand railing and the top boom is planked over on each side of the bridge, to form a footway for the platelayers. Each cross girder rests in four angle-iron gutter-shaped stirrups, each riveted to the channel iron of the top boom by six $\frac{7}{8}$-inch rivets. By these stirrups the top roadway is suspended from the main girders.

The roadway consists of three planks bolted down to two rail-bearers extending longitudinally throughout the span, one on the inner side, close to the bottom boom of each girder, resting on and riveted to the cross T irons, which are riveted to the underside of the distance pieces in the boom.

The hand railing (Fig. 7) is of flat wrought-iron bars, formed into hexagonal figures fitting together, and riveted with an angle iron top and bottom, the bottom one carrying the ends of the small cross planks forming the footways. Over each pier the hand railing is recessed outwards to form a small refuge for platelayers during the passage of trains.

The arrangement for expansion consists of a flat plate, secured by studs to the bottom plates at the ends of the girders, sliding on a wrought-iron bed plate. The girders are fished together throughout, but the holes in the end plates of the girder are slotted, and allow of the girder expanding $1\frac{1}{2}$ inch lengthways.

The bed plates are 5 feet 6 inches long across the width of the pier, and 3 feet 1 inch wide. They rest on bed stones 1 foot 8 inches thick, with an area of 33¾ superficial feet. The bed plates are secured to piers each by two long bolts going through the bed stones 6 feet into the brickwork, an arrangement which gave trouble in building without insuring any corresponding advantage.

The weight of the superstructure is—Ironwork, 2,120 tons; timber-work, 270 tons; total, 2,390 tons, equal to 0·49 ton per lineal foot.

When the operation of erecting girders was started, the river was divided into six channels. The first, about 500 feet wide (Fig. 5), with a depth of 12 feet, ran under the right bank. No. 2 was a branch thrown off from No. 1 by a sandbank at piers 6 to 10. It became eventually the main channel. No. 3, an offshoot of No. 2, ran from it nearly parallel with the bridge for about 700 feet, and then through spans 21 and 24. This channel was developed during the rains. At the commencement of operations in the cold weather it ran deep and strong between spans 21 and 23. In its course parallel with the bridge it spread out into a shallow pool. Nos. 4, 5, and 6, were shallow, having a common head at the point of crossing of the main stream of the Jhelum from the left to the right bank. These channels were reinforced by the winter rains falling on the neighbouring small range of the Pubbee hills, draining by a small river into the Jhelum.

Owing to the great depth and velocity of the water in channels 1 and 2, rendering piling difficult and expensive, it was decided to adopt means for getting the girders in place over these spans by traversing them out from the shore with the aid of a floating stage. It became important to direct as much as possible of the water in the river down the channels, with the double object of diminishing the discharge of channels 4, 5, and 6, and rendering them easy of management, and also to scour out the sandbank between channels 1 and 2, so as to admit of the boat with the stage being floated into position between all the piers. With this latter object, and to insure still water between piers 21 to 23, and 18 to 24, where it passed through the bridge, it was decided to block up No. 3 channel at its head.

Channel 4 was allowed to keep its course, but was prevented from flowing through any of the piers 20 to 24 by an embankment of earth protected by tree spurs, grass, and boulders. Channel 5 was also left open, but the discharge through it was slackened by a spur, J (Plate 5, Fig. 1), thrown across its head about 1 mile up-stream, where this spur also served the purpose of increasing the flow of water in

channels 1 and 2. Channel 6 was entirely closed by the upper spur J, and the craneway embankment running across the channel.

A tree spur at B and an embankment, A, of sandbags and earth faced with grass, thrown across the head of channel 3, completely closed it, and the sandbank between piers 5 and 9 was entirely scoured away. The embankment C, of earth also faced with sandbags and grass, prevented any water flowing through the bridge north of pier 24, and the embankments A and C together insured still water in the area between them. The upper spur greatly slackened the current through channels 4, 5, and 6, and permitted boulder islands being formed in No. 5, partial banking up of No. 4, and the complete closing of No. 6.

These arrangements made feasible the construction of a continuous craneway from the left abutment to halfway between piers 25 and 26. The craneway was formed of two sand embankments, protected with grass fascines, one on each side of the piers, and in dangerous parts of the river the intervening space was also filled up with sand. Between piers 32 and 39 the craneway was carried by small girders laid over boxes filled with boulders resting on islands formed of boulders.

The continuous craneway formed the first section of operations in the girder erection. To obviate danger from winter floods, it was thought best not to trust to riveting the girders in the river bed. They were therefore erected in a long line on high ground on the left abutment within a craneway, and when complete raised slightly from the ground by two small travelling cranes, and conveyed to span 48–49, where they were deposited and again picked up and raised clear of the piers by a Wellington crane, and by it carried down the craneway to the piers destined for their reception. In this way some girders were conveyed a distance of 3,000 feet, the mean distance being 1,800 feet. This was done satisfactorily without accident, and the plan had the advantage of permitting the work of riveting the girders being pushed on rapidly in complete safety. The season proved one of heavy freshets, and any plan of building the girders in the bed of the river would have been attended with delay, if not accident.

Carrying the craneway across the nine spans occupied by channel 5 was a difficult matter. Piling could hardly be resorted to, as the bed of the river was encumbered with boulders washed down from the armour of stones round the piers; and to drive piles through these would have been both slow and expensive. Islands were therefore formed by throwing in boulders to support the craneway. These answered very well, but great trouble was

experienced in keeping the stone boxes and girders resting on them in correct position, as every freshet undermined the islands.

Spans 13 to 24 formed the second section of operations. Between spans 13 to 19 the bed of the river was mostly above the reach of freshets, and in spans 19 to 24, the water having been rendered stagnant by the two embankments A and C, they were easily filled up with sand. The spaces between the piers were then made level, a craneway was laid down, and the girders were built in between the ways. When riveted they were easily lifted into position on the piers by a Wellington crane.

The spaces between the north abutment and pier 13 formed the third section of the work. The girders for this section were built on shore on a piece of ground in front of the shops parallel to the approach bank W, and about 17 feet lower than the tops of the piers.

When riveted, each girder was turned over on its side by a couple of derricks, and drawn up an inclined stage to the top of the approach bank, reduced in height to the level of the tops of the piers. It was then again raised by a couple of shear legs into a vertical position, whence the girders of the first span were one at a time propelled by rollers over the abutment, and was brought to bear on a stage built in a large barge anchored between the piers. When all was properly arranged, this boat was warped across the span until the end of the girder rested on the adjacent pier. One span of the girder being thus fixed, the remaining girders, which had been raised on to the approach bank, were similarly propelled one by one over rollers into a position between the pair of girders first fixed, where each was picked up by a pair of small over-head trollies working on the top booms of the fixed girders. The girder and trollies were then moved out by a chain worked by a winch on a leading pier until the girder projected sufficiently to rest on the floating stage to which it was fastened, and being released from the leading trolly, the barge was warped across the span, until the end of the girder rested on the pier. These operations were repeated for each girder, until they had been all telescopically made to traverse out. Each time a span was fixed, the lead of the succeeding girder became longer; but the over-head trollies carried the girders smoothly. The tension on the girders through the chain required to move them out was moderate, and the operation was conducted with ease and safety. When the girders had been got out from the abutment to pier 14, there only remained span 24–25 to deal with. To this the boat stage was shifted round, and the girders got across from

the adjoining north span in a similar manner. This span was purposely left till the stage boat was available, as it was not considered desirable to stop the water flowing through it by a craneway. It and half of the adjacent span, 25–26, remained quite open during the whole of the operations, and relieved the discharge of water which otherwise must have passed through the spans in which the boulder islands had been formed in section 1. By the processes above described the fifty spans of girders were erected by the 17th of March, or within three months after the order for the work had been received.

The complete riveting of the bridge, and fixing the underway and top footways, occupied till the 23rd of August. The time occupied, from the 17th of December to the 23rd of August, was thus eight months and one week for the erection of the ironwork of a bridge nearly 1 mile in length, a result probably unequalled in rapidity on any other work yet carried out in India.

On the 6th of September the bridge was tested by Colonel Pollard, Consulting Engineer to Government. Ten spans were subjected to the stress of a train of thirty loaded trucks and three engines, one weighing 25 tons, the other two each 16 tons, and each having a weight of 6 tons on the driving axles. The train was brought to a stand with the engines in the centre of each span to be tested, and was finally run over them at a speed of 15 miles an hour. The maximum deflection was $\frac{5}{8}$ inch, and the maximum oscillation $\frac{1}{8}$ inch. The bridge was opened for public traffic on the 14th of September, less than nine months from the date of beginning the erection of the girders, work having been commenced late in the cold working season, and carried on throughout the hot weather and rains.

The cost of the fifty spans of ironwork, delivered at the bridge, was £39,401, or £788 per span. The gross weight of a complete span was 41·2 tons, so that the cost per ton delivered was £19. The cost of repairing the ends of the struts, and of adding strengthening plates at the joints of thirty-two spans, amounted to 30,000 rupees. These plates added $66\frac{3}{10}$ tons to the total weight of the ironwork, or more than 2 tons per span. The cost of the timber planking in the lower subway and upper footways, amounting to 15,167 cubic feet, was £3,317, or at the rate of 2*s.* 8*d.*, or 4*s.* $0\frac{1}{4}$*d.* per cubic foot, including tarring. The cost of erecting and painting the ironwork, inclusive of all incidental charges, was £11 9*s.* per ton.

The primary cause of the high cost of erection—high even for India—was probably due to the complicated nature of the design,

and the immense quantity of riveting which had to be done at the time of erection, instead of at the manufacturers' works, as would commonly be the case with ordinary girders, as, for instance, those used at the Chenab. Secondly, to the delays which occurred in consequence of the failure of the struts as received from England. This accident led to a great deal of expenditure in preliminary arrangements, which when work was stopped was useless, and had to be incurred afresh in the following working season. Besides, the cost of work was enhanced by introducing the additional connection plates for giving the necessary strength where the ends of the struts were cracked. The total cost of the ironwork, fixed and painted, was thus £66,853; the total weight was 2,120 tons, and the cost per ton £31 10*s.*

The expenditure has been 1,530,000 rupees, which at 1*s.* 9*d.* to the rupee represents £133,875. This is exclusive of establishment charges and the cost of tools and plant. The cost per span has thus been £2,677, and the cost of the bridge, taking the length between the abutments at 4,867 feet, has been £27 9*s.* per lineal foot. The expenditure on protective works in connection with the bridge has been £5,620, bringing up the gross total expenditure to £139,502, or £28 11*s.* per lineal foot.

ABSTRACT of the COST of the BRIDGE.

—	Cost.	Total.	Percentage of Total.
Piers and abutments—	Rupees.		
Well-sinking	204,381	674,977	42·34
Concreting wells	32,136		
Brickwork	234,921		
Bed stones	11,302		
Boulders around piers	20,473		
Earthwork	84		
Well curbs	171,680		
Ironwork—			
50 spans	764,037	774,612	48·59
Small girder over abutments	10,575		
Timberwork.	37,917	37,928	2·37
Metalling	11		
Contingencies	42,568	42,568	2·67
Protective works	64,228	64,228	4·03
		1,594,313	100·00

The ironwork was designed in the office of the Engineer-in-Chief, Mr. Lee Smith, M. Inst. C.E., and was supplied in England

by Messrs. Campbell and Johnstone, of London, and Messrs. Brassey and Co., of Birkenhead. The rest of the structure was designed in India, and carried out without contractors, under the superintendence of Mr. A. Grant, M. Inst. C.E., Engineer-in Chief, Mr. H. Lambert, Superintending Engineer; and for foundations Mr. W. J. Galwey, M. Inst. C.E., Executive Engineer, and Mr. H. L. Monk, Assoc. Inst. C.E., and Mr. P. T. S. Large, Assoc. Inst. C.E., Assistant Engineers; and for superstructure Mr. F. M. Avern, M. Inst. C.E., Executive Engineer, and Messrs. Younghusband and Gerrard, Assistant Engineers.

The Paper is illustrated by several diagrams, from which Plate 5 has been compiled.

[Mr. W. H. Barlow,

Mr. W. H. BARLOW, Vice-President, desired to call attention to what seemed to be a remarkable feature, namely, the piling of a large number of concrete blocks around each pier of the Alexandra Bridge. It was stated that those piers were founded on a sandbank; but it would appear that such a mass of blocks would lead to an excessive scour between the piers, and would defeat the object which seemed to have been suggested. He noticed an indication that the blocks had settled, especially on the upper side of the pier. Had the Author been present, he should have asked him to explain the reason for using such a large number of concrete blocks, which seemed to be rather dangerous than otherwise.

Mr. WILLIAM HARVEY said he had been engaged on the Ravi Bridge, but not on the Alexandra; he believed, however, that the beds of the two rivers were very similar. Concrete blocks were placed round the piers of the Ravi Bridge, even when the piers were on high sandbanks, in the same way as in the case of the Alexandra, the reason being that when the floods came down suddenly, it was as likely that they would charge the sandbank, as that they would go through the channel made by a previous flood; therefore it was always necessary to pile up blocks for fear the flood should charge against any pier. It was impossible, before the flood came, to know against which piers it would go. In the account of the sinking of the piers of the Alexandra Bridge a description was given of three of them having been overturned before any blocks had been placed around them. The piers were originally formed on sandbanks, and it was not thought that even if the flood did come down they would be in danger; but the flood, having been turned by some obstruction in the river higher up, hit the sandbank, and where previously there had not been more than a depth of water of 1 foot, moving at 3 feet per second, there was during the height of the flood a depth of 50 feet of water, moving 9 or 10 feet per second, and the flood overturned the piers although they were formed of three wells sunk 70 feet in the sand.

Mr. LEE SMITH observed, that as his connection with India and the Punjab had been severed for many years, he had not intended to speak, but as his name had been mentioned in terms of disparagement, he would say a few words in mitigation of the censure which had been passed upon his design. He should not like it to be supposed that the diagram exhibited represented the outcome of all his ideas. A great many designs had been prepared before one was selected. When he was appointed in 1868, orders were given that no time was to be lost. Knowing that the bridges, which were to be 17,300 feet long, were pre-eminently the key to

the opening of the line, he begged, before he went to India, that he might be allowed to prepare a design. That was agreed to by the Secretary of State, and he accordingly prepared a design which was approved by a high authority in England. The only fault found with it was, that too high a quality of metal had been specified—Howard's rolled links, and certain luxuries of that sort. On arriving in India this was all changed, and the cost was directed to be cut down to the lowest possible limit. He was told that the bridges were to be cheap or they would not be built at all. A design was therefore prepared according to what he thought was the cheapest plan, and he believed the Ravi and Jhelum bridges, fulfilled the conditions specified. He observed that the engineer in charge of the Jhelum bridge found fault with the design, and stated that it was complicated. He was sorry the Author was not present to discuss the matter, as in his absence he could only surmise that some pretext had to be found to account for the fact that the ironwork cost £1 12*s.* per ton more for erection than identically the same bridge at the Ravi. He had endeavoured to analyse the cost of the bridges, but the conditions were not given in the same manner; for example, the price of the Ravi bridge was only given per lineal foot; but adding the 5 per cent. for tools and plant, and 15 per cent. for local establishment, and also £5 per ton for rise in the price of iron, so as to compare them on equal terms, he found that the cost per lineal foot of the ironwork in place was at the Ravi £17 9*s.*, and at the Alexandra £22 13*s.*, showing a difference of £5 4*s.* The difference in cost of the finished bridges per lineal foot was still greater, and was much more in favour of the Ravi and Jhelum bridges as compared with the Alexandra bridge; but taking even the lesser figures of the superstructure alone, and multiplying £5 4*s.* by 17,300, the length in feet, it would be seen that that saving was worth effecting. With reference to the criticisms, he could only say that he was bound to carry out the orders of the Government, and to make the bridges in the cheapest form. A telegram was sent from India stating that the bridges appeared to be weak even for the narrow gauge, and he was asked to explain the circumstance, which he believed had been done satisfactorily. He proved that the conditions laid down for the rolling-load were different from those which ordinarily obtained in England. His orders were that the rolling-load was to be about 16 cwt. instead of 1 ton to the lineal foot; and the conditions laid down by the Board of Trade as to the strain put on iron were to be adhered to. The designs corresponded with all these requirements, and it would

be found, from the test to which the Jhelum bridge had been subjected, viz., a rolling-load 2¼ times more than that for which it was designed, that it stood this test with a permanent set of only ⅛ inch. He had only further to add that he had not seen the original drawings for six or seven years, and did not know that the design was his until he received an abstract of the Papers. Nor had he ever seen a single rivet or bar of the iron of which the bridges were constructed.

Mr. Gregory, C.M.G., Past-President, said it was only due to Mr. Lee Smith that he should add a few words with regard to the criticisms made on the design of the bridges over the Ravi and the Jhelum. After Mr. Smith's designs had been made, the Secretary of State for India requested him, certainly in no unfriendly spirit, and with Mr. Smith's consent, to examine the designs and report upon them. His instructions named the conditions of load which the bridges had to bear, and which were below those which he as an engineer should have assigned to them. He knew that he was not expected to criticise the design as one might criticise the work of an assistant, or to make any capricious alterations. Taking the design as a whole, it certainly appeared to him to carry out the work for which it was intended in a practical and mechanical way. No doubt he could have varied the details, and if the work had been designed under his own direction the details would have been somewhat different, but he did not think it necessary to recommend any change. With regard to the strength of the bridges and their fitness for the work, he had had the strains all carefully calculated, and it was found that the design was sufficient for the load it was proposed to bear, as was proved by the small amount of deflection produced by a load much in excess of that for which it was designed. He could not help wishing that the Author of the Paper on the Jhelum bridge had been present to explain the ground of his complaints. The design was spoken of as being defective, owing to the amount of riveting which had to be done at the time of erection instead of at the manufacturer's works. A considerable amount of riveting on the ground was necessary in a bridge of that sort, when the parts had to be transported over a country where large girders could not be carried. He was not aware, however, that there was any excessive amount of riveting, for as far as he could remember there was not more than he had allowed in bridges designed in his own office, which had been satisfactorily executed in countries somewhat similar to India as regarded labour and transport. Besides the general objections to

the design, there were objections to some of the details, which, not having the drawings before him, he was not able to explain. To a certain extent he thought the objections contradicted themselves. The Author of the Paper on the Ravi Bridge, who knew a great deal about iron bridges, had erected a similar bridge at much less cost, and had not found fault with the design. The Author of the Paper on the Jhelum Bridge complained that the diagonal struts, formed of T iron, had the web cut off near the top and bottom booms. As the reduced section was strong enough where it came near the booms, he did not think there was anything unworkmanlike in cutting off the web where the strut had the least amount of work to do. The Author at first believed that the cracking of the iron through the rivet-holes was due to the quality of the iron, from the fact of nearly all the cracks occurring in the work supplied by one firm. This seemed to be a rational conclusion, and why it was abandoned he was at a loss to understand. He was still more puzzled in endeavouring to follow out the reasoning of the Author, when he stated that because the iron stood a good test for tensile strain, it was evident that the defects were not owing to bad material. Iron might stand a good tensile strain, and yet there might be something in the material or make which would cause it to crack at the rivet-holes. The Author further stated, "It thus became apparent that the defects were not owing to bad material, but to shearing away of the web, and subsequent bad packing, and consequent damage from handling in the course of their transit from England" (*ante*, p. 100). Thus after admitting that nearly all the cracks occurred in the work supplied by one firm, and that one cause of failure was bad packing, and consequent damage from handling, he came to the most illogical conclusion that "these evils must . . . be attributed rather to the designer than to the manufacturer or to the transport agents." Why the designer should be made responsible for bad packing, and for cracks, nearly all of which appeared in work coming from one factory, and not in work coming from another, he was at a loss to understand.

Mr. G. B. Bruce thought the main interest attaching to bridges over rivers in India lay in the foundations. The piers themselves were sunk to a great depth, about 70 feet, through sand or silt, and round about them were heaped large blocks of concrete, stopping up the waterway to a great extent. The effect of the scour was such that the blocks by degrees fell into the hole, and formed a bed across the river. He agreed with Mr. Barlow in thinking that was not altogether a wise procedure;

and it might be interesting to draw attention to the different modes pursued in various parts of India. In Madras there was not so great a depth of water, and perhaps the plan pursued there might not be suitable in cases like those described in the Papers; the piers were not carried down to anything like the same depth; but on the low side of the bridge, and parallel to the centre-line of the bridge, a wall was run across the river to a certain depth below the bed (where it could be got at in dry seasons) to allow for the scour. A wall of brickwork, or masonry, or wells, would prevent the water from scouring below a certain depth. If the blocks of concrete referred to had been laid in that way, perhaps on both sides of the bridge, but certainly on the low side, it would, he thought, have been a more efficient mode of guarding against scour. There was nothing to prevent a scour to any extent between the heap of blocks round one pier and the heap of blocks round another. The plan he had referred to was the old native method always pursued in Madras, and he thought it on the whole the cheapest, and, in the end, the most economical. He could quite understand engineers, on finding that bridges with wells carried down 40 feet toppled over, as some did in the north of India, resorting to the plan of sinking them 70 feet; but he thought the object might be attained more cheaply and efficiently by a platform across the bed of the river.

Mr. TURNBULL remarked that the officers of the Government of India formerly used rectangular foundations with compartments in them, and they were sunk in large blocks about 15 feet by 20 feet. The great works on the Ganges canal were done in that way. One remarkable work, the Solani aqueduct, the opening of which he had witnessed, was constructed in that manner. The chief objection to it was that excavating the sand naturally formed a cylindrical hole which made the huge blocks hang from the corners, and consequently a fracture often occurred; otherwise the method had proved very efficient. On the East Indian railway the native system of sinking wells or cylinders had been followed, beginning with small ones, 12 feet in diameter, and ending with large cylinders, 20 feet in diameter. He agreed with Mr. Bruce as to the advisability of spreading the blocks over the surface, instead of heaping them about the piers, but each case should be decided by its own peculiar conditions.

Mr. W. FURNISS POTTER said he had had several years' experience on the Madras railway, and he agreed with the remarks of Mr. Bruce as to protective works for foundations. He had himself built a number of those works for bridges in Madras, and they

had been extremely efficient. A wall was sunk a given depth in the sand, and built across from pier to pier, and then a good solid flooring was put underneath the bridge. There was, however, one danger to be guarded against. The flooring and the top of the wall should be a considerable depth below the existing bed of the river, otherwise the river would be likely to scour, and the bed might be lowered so that an embankment would be formed across the river by the protective works, which would occasion still further scour and be a source of great danger. He had seen works that had not been built sufficiently low, and that eventually had to be lowered. There was no doubt that, if they were built low enough—5 or 6 feet below the ordinary bed of the river—they formed an excellent protection against the scouring of foundations.

Mr. Elwes said the Author of the Paper on the Jhelum bridge had remarked that "most of those engineers who have gained experience of well foundations in these peculiar rivers are now agreed that one large circular well would be the best of all foundations, both as a matter of security, and on account of ease and rapidity in sinking" (*ante*, p. 99). He had been acquainted with those rivers for seventeen years, and during eight years of that time had been engaged in operations connected with them, and he was anxious to support the opinion of Mr. Avern, because it agreed with all he had observed. He was the more anxious to do so, because the system of a single well had perhaps suffered somewhat from the failures of the Sutlej bridge, where the wells were 12 feet in diameter, and sunk to a depth of 40 feet. In that case, he thought the failure was occasioned mainly by the diameter of the well not bearing a sufficiently large proportion to its height; and he observed that the same opinion was expressed with reference to the overturning of the wells of the Alexandra bridge. In the river Tangree, near Umballa, he had sunk a well of 36 feet external diameter, and 30 feet clear internal diameter, to a depth of 33 feet below the bed of the river, and he had rather less difficulty in getting that well down than he had often experienced in wells of a third or a fourth of the diameter. As to protecting foundations with a curtain wall and floor, he might mention that the Markunda bridge in Northern India had been built expressly as a trial of that system. The cause to which he attributed the partial failure in that case was, that the protection on the lower side of the bridge was formed by a row of wells 5 feet in diameter, with spaces of from 15 to 18 inches between them. Those spaces no doubt should have been made watertight by piling.

It was originally intended that there should be two rows of wells, one row opposite the intervals between the other row; the second row, however, was left out for economy, and it seemed to have escaped notice that there was a danger of the water finding its way under the flooring, and through the spaces between the wells, so that the line of wells ceased to be an impervious barrier. In consequence it was found necessary to sink a second curtain wall lower down the river, and to connect it with the existing floor by an apron of concrete, which he believed had been successful. The bridge was completed in 1867, and he had heard no complaints since of any danger to the foundations. The Solani aqueduct was remarkable, because the fall of the river was great, and the waterway was contracted. The curtain was formed of square blocks with spaces between them, which were filled up with piling. Where wells were used for a curtain wall in preference to rectangular blocks, he would suggest that it might be better to adopt an hexagonal well instead of a round one, so as to give greater facility for piling between, and stopping the water from finding its way under the flooring, as engineers who had driven piles between two circular wells of small dimensions knew how difficult it was to make a watertight joint.

Mr. IMRIE BELL said as the opinions expressed on the Papers seemed to vary, perhaps his own experience might not be unacceptable. The training works described in the Papers seemed to have been suitably designed and well carried out. In India, where the banks were low, there was a risk of the water getting behind the protective works, and bursting through the embankments, unless they were carried far inland into high ground. When any exceptional rise occurred, as during the monsoons, the river would spread over a breadth of 2 or 3 miles, and sometimes, if it met with an obstruction, it formed a channel entirely away from that of the previous year, and would, in the case of a railway, cut into the bank, and sever the connection from one side to the other. If the protective spurs or groynes were carried some miles into high ground the difficulty would be obviated. In former times engineers protected the piers, against which the current of the river during heavy floods appeared to be causing a dangerous scour, by putting stone round them; but they were not designed for that, as seemed to have been the case on the State railways, where concrete blocks were made and stone collected ready to be placed round the piers when they were built. He had bridged the Jumna twice; and at Allahabad in one year, when the river rose very high, there was only one of the piers that required to be

protected with rubble thrown round it. Those piers were formed upon foundations of ten wells, four on each side, and one up and one down stream; the stone thrown round the pier saved it, but that procedure had never been contemplated. It appeared to him to be a clumsy and dangerous plan to trust to throwing in rubble and concrete blocks, which would increase the scour and be liable to divert the current against one particular pier, and that the proper way would be to form the foundation upon a well, or on two or three wells of large diameter; because, even in the bridges described in the Papers, experience had shown that there was a want of base when a pier was 70 feet high and only 12 feet 6 inches in diameter, and he did not wonder at its going over. The least diameter ought to be one-third of the height, whether in the case of one or two wells; but if there were a series or group of wells, as in the Alexandra bridge, they ought to be securely tied at the head of the foundation. The kunkur lime used appeared to have had too large an admixture of charcoal ash. He had employed it with a small portion of clay so that it could be made into bricks, be placed in a kiln and then burnt. Afterwards the bricks were taken out without any mixture of the ash of the material used for burning. The clay was necessary in mixing the mortar to make it an hydraulic lime. By making the kunkur lime into bricks in that way there was the double advantage of not deteriorating it with a mixture of wood or charcoal ash and of getting it crushed pure.

Mr. Brunlees, Vice-President, said he had had considerable experience in the erection of viaducts and piers in sandy estuaries, and his practice had been to run a weir of stone across the base of the structure at a level corresponding with the natural depth of the ordinary channel. Whenever any obstacle had been placed higher than that level he had always found mischief to accrue. So delicate and tender had been the sands with which he had had to deal, that if an obstruction had been placed upon them, he did not suppose the works erected would have stood one spring tide.

Mr. Harvey, in reply to a question as to the alleged complicated design of the girders of the Jhelum bridge, said he believed Mr. Avern referred especially to the difficulty of getting in the cross girders. It was impossible to put some of the cross girders in till after the stirrups, or straps for supporting the cross girders of the upper roadway, were in their places; therefore all the stirrups of each girder had to be first riveted up. This necessitated one main girder being shifted to one side and then worked back again on to the cross girders, which had in the meantime been

temporarily supported at one end after the other end had been pushed through the stirrup of the other main girder as far as possible, the reason being that the blocks connecting the stirrup straps were fastened by rivets which could not be got at when the cross girder was in place. With reference to the extra cost of the erection of the ironwork of the Jhelum bridge in comparison with that of the Ravi bridge, he thought it was only fair to say that it might be due to the fact that that bridge was 100 miles farther north than the Ravi bridge; and that the lead of that hundred miles was included in the cost of the erection. It had been suggested that the bridges were originally designed to have protective works round the piers, but that was not the case. On the Punjab and Delhi railways, bridges over similar rivers had one well to each pier. It was not thought at first that protective concrete blocks would be required in any quantity even for this description of pier; but when it was found that the piers were washed away blocks were put in. On the Punjab Northern railway it was considered that three wells sunk 70 feet would be safe; and it was only after a scour had been noticed to within 20 or 25 feet of the bottom of the wells, sunk 70 feet, that it was deemed necessary to put in any blocks. The idea of the blocks was to create a scour; they were not put to stay where they were; the object was that the water should come against them and sink them down, so as to form a continuous flooring across the stream. The toe of the slope from one pier should run out towards the toe of the slope from the other pier.

Mr. Benedict thought the block system, though it might be rough, was very effective. Its roughness ought not to militate against it any more than in the case of a breakwater. It did its work, and that was the great thing. As to causing an obstruction, the blocks were not put in the river until there was a scour. A scour meant that the river bed had got bigger, and the object of the blocks was to restore it to its original dimensions. The blocks had not been put in as he should have put them in. His experience had been that round a cylinder in a sandy and silty soil a funnel-shaped hole was formed. The proper way was to wait until that hole was formed, and then to put the blocks in. It was necessary to sink the cylinder low enough to be safe for one season. If the scour was greater than was expected it might be needful to pitch in stones in the first season, but the chances were that it would not be necessary. He wished to mention one way of protecting banks, which, after all, was the great difficulty. He had adopted it successfully at the Gorai bridge, where there was a rise

and fall of 30 feet of water, a depth of 50 or 60 feet at low water, and a stream flowing at the rate of 5 miles an hour. Recesses were cut into the bank and filled with bricks thrown in loose. At the Gorai bridge these recesses extended 100 feet back from the edge of the water; they were 30 to 40 feet broad, 30 feet deep, and were from 200 to 500 feet apart on the concave shore of the river.

Mr. P. T. S. Large observed, through the Secretary, that he had been engaged on the Jhelum bridge from its commencement in 1870 to April 1874. During the season 1870–71 the only work taken in hand was the preparation of materials. The actual construction was commenced rather late in the season of 1871–72. The season each year commenced about the 1st of October, and ended when the river rose in April. Ten piers, consisting of thirty brick cylinders, were put in hand on the south bank. They were carried on wrought-iron curbs, which, after being placed *in sitû*, were filled with concrete and rammed. By order of Government these cylinders were pitched 6 inches apart, three in each pier. This distance was subsequently found to be too close, it being difficult to keep the wells from getting foul of each other. The distance between the wells in all the other piers was 2 feet. The brickwork was 3 feet 3 inches thick in the 12-feet 6-inches cylinders, and 2 feet in the 10-feet cylinders. During the season 1871–72 the sinking was principally effected, when in sand, by the native jham, with the working of which all Indian engineers were familiar. The sand-pump also was used, but proved slow, clumsy, and more expensive in working. In consequence of the scarcity of plant the sinking this season proceeded but slowly, and was not completed when the rise of the river in May stopped work. All the cylinders had been sunk some feet into the boulder stratum, except those in the outer pier, which were unfortunately lost. When the river again subsided, the position which the cylinders occupied on the boulder bed was ascertained by careful borings. In the next season, 1872–73, Mr. Galwey, M. Inst. C.E., was placed in charge of the work. He determined to commence the sinking of the wells for the piers on the North bank, with the Author as his assistant, and with Mr. Monk, Assoc. Inst. C.E., on the South bank. The river was in a most unfavourable state, the dry season channel having widened considerably. Numerous training works of tree-spurs, or groynes, were put down at various places upstream, which acted efficiently in throwing the stream into mid-channel, partially silting-up the deep channel alongside the North bank. These

groynes were constructed of trees bound together at the trunks by a continuous rope fastened by stakes to the bottom, and weighted with nets containing boulders to keep the whole down. Being placed at an angle with the direction of the current, they had the effect of causing silt to accumulate behind them, thus diverting the course of the river. The North bank channel being thus only partially diverted, islands of sand, protected with sandbags, had to be constructed, for pitching the curbs on, for the first six piers. But the river being 15 feet deep at the site of No. 1 pier, and running with a velocity of 4 to 4½ miles an hour, this island gave considerable trouble. It was not begun until piers Nos. 2, 3, 4, and 5 had been sunk into the boulder stratum, when the river was again diverted through these spans, and No. 1 island commenced. A strong groyne of trees and nets filled with boulders was thrown out from the solid bank above the site; and when the velocity of the water at the site slackened the island was formed of sand, with sandbags and nets of boulders as a protection to the sides; grass in bundles was also put down on the outside, to prevent the sand from leaking out. During this year temporary bridges were constructed, workshops built, and a system of tramways laid down. The three cylinders of the North abutment were sunk almost entirely by means of centrifugal pumps, the clay being dug out by hand; but subsequently the sinking was completed by divers, in consequence of a boring pipe piercing the clay stratum into a sandy one beneath, and the inability of the pumps to keep down the water, which rose in large volumes through this hole. In the season 1873–74 the sinking of the cylinders for the remainder of the piers, Nos. 13 to 40, both inclusive, was taken in hand. The work through sand was almost entirely performed by Bull's Excavator. The excavation in boulders was carried out principally by divers and specially constructed sand-pumps. The whole work of sinking, concreting the wells, and building the superstructure to the level of the bolts for the bed-plates, was completed by March 1874.

Mr. E. J. Jones remarked, through the Secretary, that it had been stated that, in order to remove the island in the Wuzerabad reach of the Chenab, three cuts of varying dimensions were made, to ascertain whether the admission of a current would not widen the cuts by undermining their banks, and so hasten the demolition of the island; also that the result proved a failure. During the construction of the headworks of the Agra Canal it was found necessary to remove a great portion of an island in the Jumna; the method adopted was similar to that described by Mr. Lambert.

Four cuts, each 20 feet wide, were made through the island, and excavated to low-water level. When the river was in flood, a few men armed with crow-bars were placed along the edge of the cuts, and instructed to assist the river in widening them; this was done by driving the crowbars into the bank at any point against which the river was surging. By shaking the crowbar the earth was loosened and fell, generally in large lumps, which were soon swept away by the stream. In this way the removal of the island was effected, as far as was desirable, in the course of three seasons, at a low rate, the cost, on the average, not exceeding 4 annas per 1,000 cubic feet. The island consisted of sand, with belts of stiff clay interspersed, varying in thickness from 3 inches to 2 feet. The groynes, or spurs, constructed on the Jumna were composed of the sand excavated from the river bed and covered with clay, the slopes being protected partly by grass and partly by fascines. Two of the spurs, which were more exposed than the others, had their slopes pitched with rubble-stone, 2 feet in thickness, which was continued below the water level; they had answered the purpose for which they were intended in the most satisfactory manner; and, after withstanding the action of the floods for three seasons, were in perfect order, needing no repairs.

Mr. R. T. Mallet stated in reply, through the Secretary, that the higher cost of erection of the Jhelum girders was doubtless due partly to longer land carriage, but chiefly to the addition of a quantity of joining plates and the involved alteration of parts ordered by Government, with a view to remedy the supposed defects of the ends of the diagonals described by the Author. This had not been done at the Ravi. As attention had been drawn to the absence of criticism on the design of the girders of the Ravi and Jhelum bridges, he felt bound to express his concurrence with Mr. Avern's opinion of its complexity. It was to the great number and variety of form of the parts, more than to the shortcomings of the contractors, that the unusual amount of drifting, riming, and chipping that had to be done was to be attributed. He believed the proposal for a continuous floor across the river bed had been considered, before adopting the plan of heaps of blocks round each pier, and abandoned, as it was feared that should failure occur at any one point the concentration of the stream there would in a few hours lead to disaster. As a matter of fact, this concentration of the stream through a few of the spans was what had occurred in all the Punjab rivers. The stone or concrete blocks round a few of the piers got deeply underscoured and sank. The stream spending itself through the main channel

thus formed was unable to underscour the blocks round the other piers, and thus one small portion of the bridge remained particularly subject to failure. It would be impossible to wait till the scour occurred, and then to throw in the protecting material. In the Punjab rivers wells sunk 70 feet, or perhaps twice that depth, might be scoured out before they could be protected. While the flood was running strongly the blocks could not be thrown in. Barges could not be brought or kept in position above stream; and if the blocks were thrown in from the bridge itself, they would be carried far away by the current before they could reach the bottom. If the subsidence of the fresh were waited for, it would always be found that the scour-hole had become already filled with sand. It would be seen that the problem of pier protection, under these circumstances, was not an easy one to solve.

Mr. H. LAMBERT observed, through the Secretary, that it was intended that the blocks should settle and disappear below the bed of the river at each pier, as it was attacked by the swift moving body of water in the main channel, which was in a state of constant change. The Government of India had been advised that the use of large quantities of blocks and stones constituted the most certain means of protection for piers under such circumstances, and the work was executed in conformity with their orders. The blocks and stones had settled, as expected, wherever the main body of water ran during each flood season, and they had been continually added to by fresh material thrown in from above. In the case of the overturned wells, blocks would have been placed around them, although in a comparatively safe position, had there been time before the river rose. No comparison could be made between a tidal estuary and the Chenab river during a monsoon flood; and it was only during such floods that dangerous scours occurred, and then the piers were inaccessible by boats. Hence, if it were decided to wait until a hole was formed, the pier might be destroyed before the remedy could be applied. In the case of long bridges, where great quantities of material were required, it was necessary to lay it down during the working season, and while the river was safe. After the girders were erected stone was thrown in from wagons to replace the material which had sunk into the holes. Large concrete blocks could not be rapidly or safely handled under such circumstances, and small ones were swept away by the current while sinking. Only a limited amount of stone had been procurable. Groynes of the kind constructed in the Jumna would not stand in the Chenab,

and the latter river was too large to be affected by a few men with crowbars.

Mr. Avern explained, through the Secretary, that he had not meant to disparage Mr. Lee Smith's design as a whole, inasmuch as the bridge now erected was a very stiff one, and satisfactory in its working for mètre-gauge trains. As mentioned in the Paper, one span had been actually tested with a load equal to twice that for which it was designed, with the result of only $\frac{1}{8}$ inch permanent set. Shortly before the erection of the Jhelum girders he frequently visited the Alexandra bridge. Compared with the girders there, those of the Jhelum were complex, and the amount of riveting to be done at the time of erection very considerable. There the booms were received at the works completely riveted, in short sections, which had only to be connected with cover plates and the diagonals riveted to the booms; whilst at the Jhelum the booms were received in separate pieces, viz.: in the top boom of the girder two separate channel irons, distance pieces, and a cover plate, which extended over the side of the boom and served as a floor plate, and three lines of rivets had to be put in at the time of erection throughout the length of the bridge, 4,875 feet. There was, besides, a set of buckle plates, also requiring a double row of rivets throughout the bridge. At the Chenab the cross girders rested on and were riveted to the top flange of the bottom boom; whilst at the Jhelum the cross girders were carried in stirrups riveted to the top boom in a way which made the putting in place of the cross girders a troublesome matter. He thought these differences of detail in design accounted for the higher cost of erection at the Jhelum than at the Alexandra bridge, or any other of like simple design. He attributed the extra cost of erection at the Jhelum, as compared with that at the Ravi, 1st., to the vexatious delay which arose from the ends of the struts cracking where the web had been cut away from the T irons. This involved the loss of a whole cold season, and what, in consequence, became useless expenditure in temporary tramways, sand banks, the collecting and subsequent dismissal of riveters and other workpeople, and expense in similar ways. 2ndly, To the cost, direct and indirect, of remedying the defects in the cracked struts by the addition of connection plates at the junction of booms and diagonals; and 3rdly, to the higher cost of riveting at the Jhelum, where $\frac{7}{8}$-inch rivets were paid for at Rs. 7 per 100, instead of Rs. 5, as at the Ravi.

May 7, 1878.

JOHN FREDERIC BATEMAN, F.R.SS. L. and E., President, in the Chair.

The following Candidates were balloted for and duly elected :—John Henry Bostock, Hugh Meller Bradford, Henry Arthur Dibbin, John Hingston Fox, Arnold Lupton, Henry Rofe, and George Lenton Roff, as Members; Edwin Addenbrooke, Alfred Allen, Jun., Frank Baker, John Bramley Ball, George Procter Carless, Stud. Inst. C.E., George Burton Chadwick, George Henry Crowther, George Workman Dickson, B.A., Charles Arthur Friend, Stud. Inst. C.E., William Greenwood, Arthur William Lawder, William Nicholl, Richard John Gifford Read, Stud. Inst. C.E., Henry Joseph Richard, Arthur Hill Rowan, Hubert August Otto Weiss, Stud. Inst. C.E., William John Wilson, Stud. Inst. C.E., Benjamin Frederick Wright, William Wright, and George Digby Wybrow, as Associates.

It was announced that the Council, acting under the provisions of Sect. III., Cl. 8, of the Bye-Laws, had transferred Thornton Andrews, Henry Percy Boulnois, Charles Ormsby Burge, Charles Copland, Frederick Eliot Duckham, Richard Gervase Elwes, Richard Hodson, Henry George Clopper Ketchum, John Lawson, John Bower Mackenzie, William McLandsborough, Frank Morris, Thomas Newbiggin, Rupert Turberville Smith, and Arthur Henry Whipham from the class of Associate to that of Member.

Also that, under the provisions of Sect. IV. of the Bye-Laws, the following Candidates, having been duly recommended, had been admitted as Students of the Institution :—Charles Henry Ashforth, Arthur Horatio Brothers, Alfred Alexander Begnigno Chester, Frederick Richard Clapham, James Gillespie Clow, John Charles Lang, Charles Arthur Lovegrove, Percy Rickard, Edgar Smart, James Hardy Southern, James Naaman Taylor, and Maurice Fitzgerald Wilson.

Paris International Exhibition of 1878.

Mr. Bateman, President, said that he had had some correspondence with M. Tresca, Member of the Institute of France, and President of the Society of Civil Engineers in Paris, and

Mr. Manby had also had frequent correspondence with him, with reference to the French International Exhibition of 1878. M. Tresca on the part of the Society offered the use of their rooms, with every hospitality and attention that could be shown to the members of the Institution who might visit Paris during the Exhibition, whether as a body or individually. He had already written to M. Tresca expressing his appreciation of the offer, and stated that the members would gladly avail themselves of it. He believed the general feeling of the members would be that although they were not likely, as a body, to visit Paris during the Exhibition, many of them would be there individually, and would be glad to avail themselves of the kind offer of M. Tresca.

No. 1,560.—"The construction of Steam Boilers, adapted for Very High Pressures." By JAMES FORTESCUE FLANNERY.

In the year 1874 the public mind was filled with alarm at the reported deficiencies in the boilers of the iron-clad fleet. That rapid decay of boilers had occurred in connection with increased economy of fuel was a fact well known to the profession, but until the time mentioned, comparatively little general attention had been drawn to the efforts made to reduce this evil, and to improve the marine and other boilers in the directions of increased working pressures and greater longevity. It is remarkable that, until late years, the skill of engineers has been, for the most part, concentrated upon improvement in the means of using, to the comparative neglect of improvement in the means of generating, steam. To so great an extent has this been the case, that it is now reasonable to suppose that the appliances for using steam have been improved to almost the highest possible degree, and yet the theoretical value of fuel is still many times greater than the result obtained in the best modern steam engine; thus leading to the conclusion that any measure for improved economy must operate in the generation of steam rather than in its application to motive purposes. It may be taken for granted, that, for the greater measure of expansion possible with very high pressures, boilers of the existing general types are useless; but, even supposing the boiler of present adoption were satisfactory as an economical generator, it is still evident that there is something inherently defective in a system which limits the life of a boiler to a period of half, or perhaps one-third, the time during which the engine may be expected to work. The evidence given before the Admiralty

Boiler Committee having been published, the time seems opportune for the discussion of this subject. It is proposed in the present Paper to refer to that evidence and to the reports of the Committee, so far as they relate to the subject under discussion.

Before proceeding to consider some of the questions connected with the generation of high-pressure steam, it is desirable to ascertain briefly what gain may be expected from its use. Much discussion upon this point has been raised, and several experienced engineers doubt if the system possess economy commensurate with its supposed evils. The use of high-pressure steam is suggested in the first instance by the fact that, no matter what the pressure, the quantity of coal necessary to generate a given weight of steam is practically the same; thus, steam at 30 lbs. pressure per square inch contains a total heat above zero Fahrenheit of 1,190°, and at 240 lbs. pressure, a total heat of 1,235°, or only 4 per cent. more heat with eight times the initial pressure. In the discussion following a Paper on this subject, read before the Institution of Naval Architects,[1] Mr. McFarlane Gray compared the relative economy of the present and former engines, showing that at one time a boiler pressure of 25 lbs., or an absolute pressure of 40 lbs. per square inch, yielded 6 HP. for a certain quantity of coal, whereas now with a boiler pressure of 65 lbs., or an absolute pressure of 80 lbs. for the same quantity of coal, the result is 8 HP.; that was to say, the pressure had been multiplied by 2, and 2 added to the 6 HP. From this comparison and from calculation, Mr. Gray inferred that "the HP. increases in arithmetical progression, and the pressure in geometrical progression." Setting this out in tabular form it appears thus:—

Lbs. pressure.	HP.
40	6
× 2	+ 2
80 present pressure.	8 present efficiency.
× 2	+ 2
160	10
× 2	+ 2
320	12
× 2	+ 2
640	14
× 2	+ 2
1,280	16

[1] *Vide* Transactions of the Institution of Naval Architects, vol. xvii., p. 279.

Fig: 6.

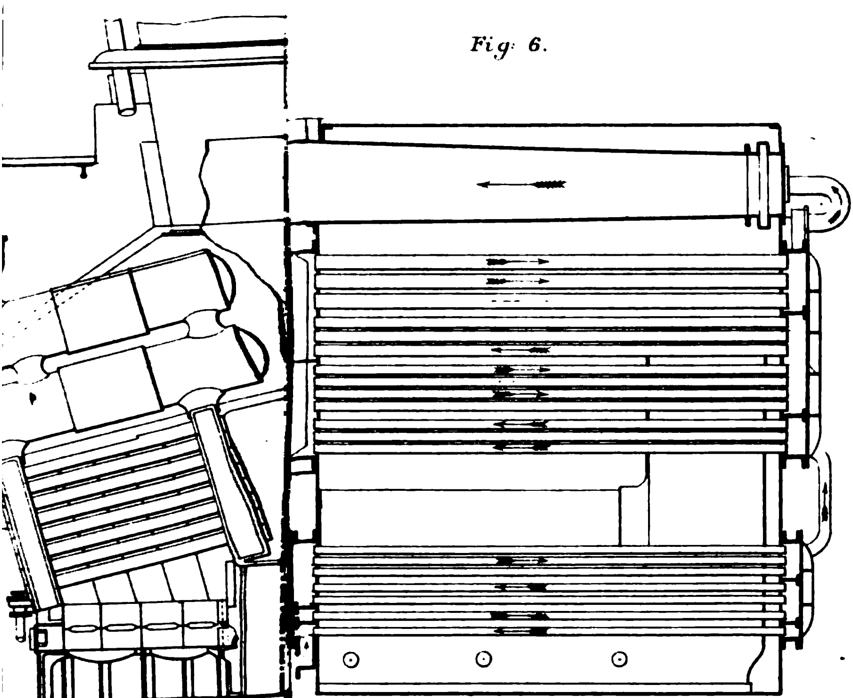
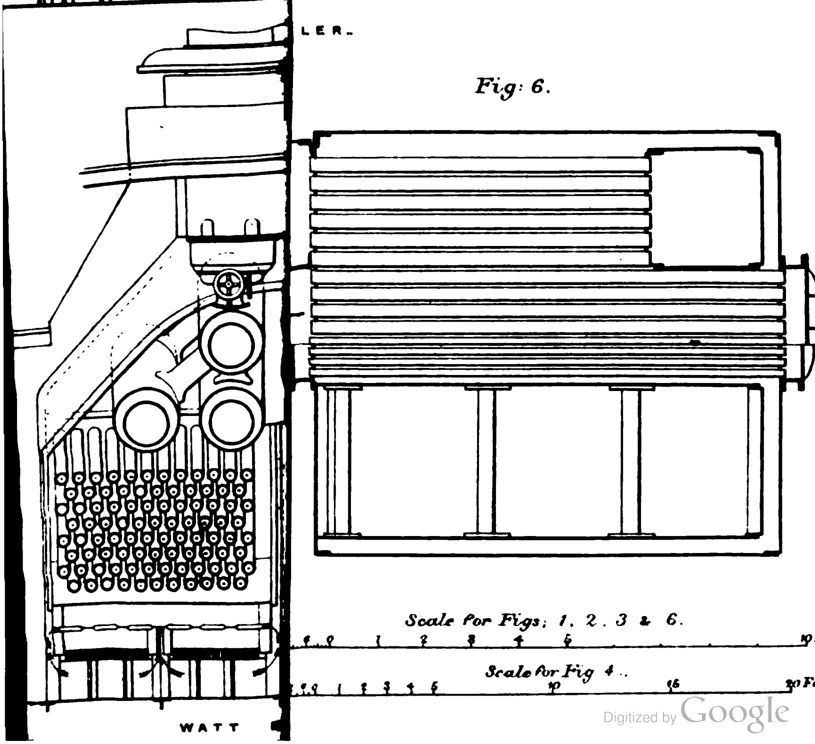


Fig: 6.



Scale for Figs: 1. 2. 3 & 6.

10 Feet.

Scale for Fig 4.

20 Feet

and the conclusion to which Mr. Gray arrived was, that "it would require 1,280 lbs. on the square inch to double the present efficiency, even if that pressure could be carried without additional drawback."

A calculation of the theoretical gain from increased pressures, with the various grades of expansion corresponding to the same terminal pressure, gave the results shown in the following table:—

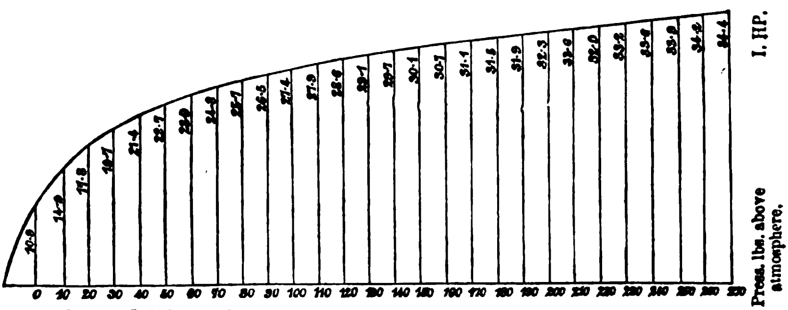
Pressure.		Ratio of Expansion.	Mean Pressure.	Indicated HP., or Mean Effective Pressure.	Percentage of Economy, compared with Economy of 60 lbs. of Pressure.
By Gauge.	Absolute.				
Lbs.	Lbs.		Lbs.		
300	314·7	39·3	37·4	35·4	48·3
290	304·7	38·1	36·9	34·9	46·0
280	294·7	36·8	36·8	34·8	45·6
270	284·7	35·6	36·4	34·4	44·0
260	274·7	34·3	36·2	34·2	43·2
250	264·7	33·1	35·9	33·9	42·0
240	254·7	31·8	35·6	33·6	40·7
230	244·7	30·6	35·2	33·2	39·0
220	234·7	29·3	34·9	32·9	37·3
210	224·7	28·1	34·6	32·6	36·4
200	214·7	26·8	34·3	32·3	35·2
190	204·7	25·6	33·9	31·9	33·5
180	194·7	24·3	33·5	31·5	31·8
170	184·7	23·1	33·1	31·1	30·1
160	174·7	21·8	32·7	30·7	28·5
150	164·7	20·6	32·1	30·1	26·3
140	154·7	19·3	31·7	29·7	24·2
130	144·7	18·1	31·1	29·1	21·7
120	134·7	16·8	30·6	28·6	19·6
110	124·7	15·6	29·9	27·9	16·7
100	114·7	14·3	29·4	27·4	14·6
90	104·7	13·1	28·5	26·5	10·8
80	94·7	11·8	27·7	25·7	7·5
70	84·7	10·6	26·8	24·8	3·8
60	74·7	9·3	25·9	23·9	..
50	64·7	8·1	24·7	22·7	
40	54·7	6·8	23·4	21·4	
30	44·7	5·6	21·7	19·7	
20	34·7	4·3	19·8	17·8	
10	24·7	3·1	16·9	14·9	
0	14·7	1·8	12·9	10·9	
	0·0	0·0	0·0	0·0	

A terminal pressure of 8 lbs. has been adopted in the calculation, because it is a usual terminal pressure in modern compound engines working at 60 lbs. pressure. The table is based upon the assumption that the expansion curve is a true hyperbola, which involves a slight error; a deduction of 2 lbs. for imperfect vacuum has been made from the mean pressure due to the boiler pressure expanded

to 8 lbs. terminal pressure, and the mean effective pressures thus found represent comparatively the theoretical amounts of HP. due to the pressure and measure of expansion. For example, with the usual pressure of 60 lbs. the result would be a ratio of expansion of 9·3, a mean pressure of 25·9 lbs., and a mean effective pressure or comparative HP. of 23·9; and for a pressure of 300 lbs. by the gauge there would be a ratio of expansion of 39·3, a mean pressure of 37·4 lbs., a mean effective pressure or comparative HP. of 35·4 lbs., and a percentage of increased efficiency of 48·3 as compared with steam of 60 lbs.

It is worthy of remark that the area enclosed by the hyperbolic curve increases in much smaller ratio at the higher stages of pressure than at the lower stages; or, if the indicated HP. derivable from each pressure be set out as ordinates to a base line, the form of the curve (Fig. 1) will graphically illustrate

Fig. 1.



Curve of I. HP. at various pressures, theoretically, obtained from a given weight of coal.
Scale—vertical, 1 inch = 24 I. HP. Horizontal, 1 inch = 80 lbs. pressure.

the fact that, while higher pressures are far more difficult to use, they are relatively of less value at the higher stages. There is at present no proof that practical difficulties would reduce to a serious extent the theoretical gain incidental to increased pressure and expansion. Assuming, however, that by increasing the present boiler pressure, and adopting suitable expansion, it would be possible to increase the economy only 20 per cent., as anticipated by Mr. Holt in his Paper on "Steamship Progress,"[1] the advantage at sea, would be of the most important kind. It may be taken as an accepted fact, that higher pressures combined with greater expansion would reduce the consumption of fuel, and it may also be believed that no improvement other

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. li., p. 7.

than higher pressures seems to hold out a reasonable prospect of increased economy. Assuming then that the advantage to be obtained from the use of higher pressures is simply a question of degree, it has to be considered by what means steam of higher pressures may be generated with safety and economy.

There can be no doubt that boilers of the existing types, especially marine boilers, will not carry much higher pressures than they are now subjected to, even though the usual factor of safety might with propriety be reduced. The limited space allowable on board ship for boiler room, and the necessity of occupying that space in the most economical manner, leave scarcely any choice as to the external size of each separate boiler; and, the size being thus dictated, the pressure to be carried is necessarily limited by the possible thickness of the iron of the outer shell, and the means of riveting it. The ultimate thickness of iron which present appliances permit riveting together as a steam-tight joint, is about 1 inch, and this thickness, for the usual sizes of shell, limits the pressure to 70 lbs. or 80 lbs. per square inch. To reduce the diameter of the vessel subjected to internal pressure is the expedient which naturally presents itself, and this reduction of diameter involves the application of the fire outside the vessel containing the water, and leads directly to the water tube or tubulous boiler.

On the subject of the tubulous boiler, the report of the Admiralty Committee says :—" The committee have made further inquiries into this system of constructing boilers, both for marine and land purposes, and the evidence has been of a very satisfactory nature. They believe that such a system of construction, combined with the exclusive use of fresh water and tight condensers, will lead to good results as regards endurance, safety from explosion, and probably economy. The chief point to be observed in working boilers on the tubulous system, exists in the necessity for supplying them with water which contains no solid matter, such as that introduced into ordinary marine boilers using sea feed. This exclusion of solid matter which may deposit itself upon the surfaces is of vital importance, on account of the small water space in these boilers when compared with those in common use, and the consequent impossibility of cleaning them; even in those where tubes of small diameter have formed only a part of their construction, the accumulation of deposit has been a serious evil. The absence of scale in this system of working introduces the important consideration of the economical production of steam, because scale has a bad conducting power, and is deposited more

thickly upon the heated surfaces than elsewhere, so that any system of working by which it can be avoided or limited in amount must command attention, and in similar cases, result in the production of steam from a smaller quantity of fuel. The successful use of either distilled, rain, or natural waters in these boilers, depends upon other conditions, such as the complete exclusion of circulating water by means of tight condensers, because the much greater pressure and, consequently, higher temperature at which they work, intensifies any chemical action upon the iron when sea, or feed, water containing air is allowed access, so that special care in avoiding its introduction becomes an absolute necessity."[1]

It is remarkable that the corrosion which shortens the life of existing boilers, attacks them in an erratic, and frequently in a most partial manner. In many condemned boilers large portions are very little corroded, although their general state may be such as to preclude the possibility of repair. This fact points to the use of a sectional boiler, so that when portions are destroyed by corrosion they may be replaced without renewing the uninjured parts. This consideration seems to have escaped the notice of many advocates of the sectional or water-tube boiler, but in the present state of perplexity it appears to be an important and additional argument in their favour.

The comparative advantages claimed for this type of boiler may be thus summed up. The diameters of the cylindrical vessels subject to pressure are much less, and the thickness of the metal for a given pressure may therefore also be much less, or a higher pressure may be carried. The relative thinness of the metal facilitates the conduction of heat; a greater margin of safety is possible in the thickness of the plates in first manufacture, and a reduced pressure is not necessitated at a later date by the wear of the metal. An explosion would be limited in area, and the smaller body of heated water contained in the boiler would again reduce the evil of an explosion. Repairs would be effected more readily; in the case of marine boilers, the necessity of breaking the decks to fit a new boiler would be obviated, as the parts could be passed down the hatchway; a ship could carry one or more spare boilers, stowed away in sections. The water-tube boiler may be so designed that the currents of heated gases will impinge on the plate surfaces at right angles, thus increasing the efficiency of each square foot of heating surface; the escape of steam will be facilitated by the

[1] *Vide* Third Report, 1877, p. xxxv.

disposition of the heating surface horizontally, as is most readily done with the water-tube boiler. It may be taken for granted that the heat primarily developed by the consumption of 1 lb. of coal is much the same in all boilers of good draught, and that, by an extension of the heating surface in relation to the grate surface, so that the temperature of the escaping gases may be reduced to a minimum, the evaporative economy of all boilers may be made nearly the same. But a boiler having favourable disposition of the surfaces will more readily be adapted to such reduction of the temperature of the escaping gases, and the maximum efficiency can accordingly be obtained in such a boiler with the least extension of the heating surface, and therefore with the least size and weight. Hence it would be difficult to overestimate the advantage in point of economy which is gained by placing the surfaces of the heating plates at right angles to the current of the heated gases. It is well known that the tendency of these layers of gas, which have parted with their heat, to continue in contact with the plate is very great, and that there is comparatively little opportunity for those layers of gas furthest from the plate, and which have not yet cooled down, to change place and part with their heat in the most direct manner, unless some means of breaking up the current of the flames are interposed. In the case of fire-tube boilers, the flames invariably travel along the heating surfaces and their disintegration takes place much less perfectly than in water-tube boilers, where the flame impinges at right angles upon successive tiers of tubes. On this ground alone it is believed that important economy may be obtained by the use of the water-tubes. Again, the conducting power of a plate of moderate thickness is, according to Rankine, much greater than the absorbing power of its fire surface, or the emissive power of its water surface, and these surfaces may, without risk of injury to the plate, be safely subjected to a greater amount of heat-transmitting work than is possible where the flame glides along the surface of the plate, but does not strike directly against it.

Another important advantage incidental to the water-tube or sectional boiler, if well designed, is its facility for expansion and contraction under varying temperatures without undue strain upon the joints. It is well known that one of the greatest evils of the present cylindrical marine boiler is the wear and tear, and ultimate leakage of the seams of the shell, from its unequal expansion; and, although such curative devices as artificial circulation, lengthened internal feed pipes, &c., have been resorted to, still the evil remains—a fruitful source of regular and costly

repairs. There seems little hope of removing this defect from the cylindrical boiler, but the sectional boiler is, in almost all cases, entirely free from it.

Against the advantages peculiar to the water-tube boiler must be set the difficulty of obtaining good circulation. The small volume of water contained by the sectional boiler, in proportion to its heating and grate surfaces, leads to fluctuation of pressure and extra danger if the feed water is suddenly stopped or the fires are forced. The circulation of the water is more rapid and therefore less safe, owing to the smaller body of water heated to nearly boiling point; and this effect is aggravated by the separate action going on in each tube. The exposure of the tubes to the direct influence of the fire, while under tensile pressure, enables the fire to search out more readily any flaws in the iron; and further, there is an alleged tendency for the outer casing of fire brick, when fitted on board ship, to work loose in a seaway, a very dangerous exposure of the flame being thus produced.

Balancing these defects of the water-tube system against its admitted advantages, there does not appear to be any insuperable objection to its general and satisfactory adoption. The difficulty of obtaining an equable circulation of the water may be met by a less tortuous and erratic disposition of the passages. The dangers arising from the smaller volume of heated water may be obviated by the use of more efficient safety valves, extra feed pumps, and regular and careful firing. The development by the fire of flaws under tensile pressure may be avoided by the use of carefully-selected iron, with the welds of the tubes placed furthest from the flame.

It is now proposed to invite attention to some of the examples of high-pressure boilers upon which these remarks have been based. The system known as Rowan and Horton's has had very complete trials, some of which have extended over several years. In more than one prominent example the boilers had to be removed; yet the inventor produces well authenticated reports of the successful working of other examples of the same type. Messrs. R. Stephenson and Co. in 1860 fitted four or five vessels with boilers on Mr. Rowan's system intended for river work in India; the indicated HP. being 430, with 3,960 square feet of heating surface, and 66 square feet of grate surface. On trial the consumption of fuel is reported to have been 1·7 lb. per indicated HP. per hour, at 120 lbs. pressure of steam per square inch. These boilers subsequently worked for ten years in an entirely satisfactory manner. A set of boilers on this plan was fitted on board a French Admiralty

despatch boat in 1861, and worked for upwards of six years at 120 lbs. pressure with great success; indeed it is said that practically no repairs were required during four years' steady working. Mr. F. J. Rowan has favoured the Author with some of the results, as regards coal consumption, of trials of these boilers when fitted for marine purposes. The "Thetis" consumed, according to an experiment conducted by Professor Rankine, 1·018 lb. of coal per indicated HP. per hour under steady working. The "Guajara," paddle steamer, consumed on trial 1·506 lb. of coal per indicated HP. per hour during regular work, as shown by records taken after the vessel had been eighteen months at work. On trial of the "Sicilia" in 1861, the consumption is said to have been as low as 1·36 lb. of fuel per indicated HP. per hour.

Fig. 1 (Plate 6) shows an improved design by Mr. F. J. Rowan, fitted with a mechanical stoker. This design differs from that of the "Propontis," on the same system, chiefly in a better provision for the return to the water spaces of the priming, in the abolition of the horizontal water chambers, leaving the generation of steam to be effected wholly by the vertical tubes, and in increased diameter in proportion to length of the vertical tubes. No doubt these modifications will prove advantageous, especially the one by which the diameter of the vertical tubes is increased; because a greater volume of water in proportion to the heating surface being contained in the tube, the tendency to priming will be less. The most serious objection to the use of boilers constructed as shown in Fig. 1 (Plate 6) appears to be that the bends at the ends of the tubes render access for cleaning difficult. If, on this account, pure water be used, the corrosion ensuing soon has a destructive effect upon the metal used, which is only $\frac{3}{16}$ inch thick. It was from this cause that the boilers of the "Propontis" failed, although their consumption was proved to be about $1\frac{1}{2}$ lb. of fuel per indicated HP. per hour, or 40 per cent. less, with a speed 1 knot greater, as compared with the performance of the lower pressure engines with which the ship had previously been fitted.

One of the best known designs for a water-tube boiler is the system called the "Root Patent." This boiler presents many advantages, and, where a large boiler in proportion to the work to be done can be fitted, it will give a good supply of dry steam at high pressures. But the twisted passages by which the successive tiers of tubes are connected present a series of impediments to the free exit of the steam that must be of the most serious consequence. When the steam is abstracted by a large and slow-moving piston, or where the reservoir of steam is small, the

steam-pipe is attached to the boiler at but a limited height above the tubes, priming and withdrawal of the water from the bottom tubes may be expected to occur. The system of connecting the tube ends undoubtedly possesses the advantages of comparative simplicity in first manufacture, and in accessibility for cleaning and renewal, and for these reasons the Root boiler has met with much success. It is believed that it could be made to carry higher pressures of steam than have yet been attempted—and, indeed, would probably work even better at these pressures; but it should in no case be used unless proportionately large space and weight are available. Specimens of these boilers were tried on board two vessels, one a ferry-boat plying upon the river Mersey, and the other a steamship of 2,000 tons. In both these cases the defects mentioned above were exhibited, and the invariably limited space available on board ship leads to the supposition that boilers of this design are suitable chiefly for stationary purposes. The Root boiler fitted in the ferry-boat referred to gave some excellent data for comparison in point of economy, because two sister-vessels, fitted with ordinary boilers and engines of the same type and ratio of expansion, were at work upon the same station, and the consumption of coal with the tubulous boiler amounted to 17 per cent. less than that of one sister vessel, and 25 per cent. less than that of the other. This economy may be wholly attributed to the effective arrangement of the parts, the thinness of the tubes, and the direct action of the flame upon them, and is of great importance when it is considered that the working pressure in this case was only 40 lbs. per square inch.

Another design of tubulous boiler which has met with much favour from steam users is the well-known Howard boiler. The design of the firm, as fitted on board ship, is shown in detail in Fig. 2. The vertical wrought-iron tube A is intended for the upcast, and the series of cast pipes B for the down-cast, the water-tubes being of wrought iron, slightly inclined from the horizontal to favour the escape of the steam; C is a reservoir communicating with each vertical range of tubes, and D is a steam-chest placed well above the water line. Doors are fitted to each tube, and are therefore readily accessible for cleaning. In practice these boilers were found to answer fairly well at sea, so long as no attempt was made to force them; whenever this was done, priming commenced, and the water was dislodged from the tubes nearest the fire, which were fractured in consequence. It is believed that if the down-cast for the water, which is only one-ninth of the area of each horizontal tube, were considerably enlarged, much improved working

would result. In a boiler of this description, the water spaces at the ends of the horizontal tubes should be of as great, if not greater, area than the cross section of the tubes, or a proper circulation of the water cannot be expected. Fig. 3 represents a high-pressure boiler fitted, by the Barrow Shipbuilding Company, to the steamship "Red Rose." It is called a Howard boiler, although it differs materially from the type generally known by that name. It is, indeed, a fire tube boiler, but so arranged that the diameters of all the cylindrical parts are much reduced, and a very high pressure for a moderate thickness of plate thus made possible. The boiler consists of three separate vertical sections, delivering steam into a common receiver, placed transversely to them. Each section is divided into three parts; the bottom, or furnace, surrounded by a cylindrical water-chamber, and communicating, as regards the water-spaces, with the cylindrical chamber above by a vertical neck 20 inches in diameter, and, as regards the flame, by a flat-sided combustion chamber, surrounded by a water space; the sections have each fifty-five 3-inch tubes. The alterations shown were required by the officers of the Board of Trade before granting the certificate. A little consideration will show that these alterations were necessary, and that the Chief Surveyor of the Board was fully justified in insisting upon them. In the first place, the cylindrical portions containing the furnace and fire-tubes are 3 feet 6 inches in diameter; and these were cut by a hole 20 inches in diameter, intended for water communication; the holes were so large, in proportion to the diameter of the shells, that the circular condition of the latter, upon which shape their strength had no doubt been calculated, was seriously interfered with, if not destroyed; the introduction of the tie-bolts shown in red is thus explained. Again, the water space immediately over the hottest part of the furnace was confined to a horizontal pocket, or *cul-de-sac*, 3 feet 10 inches long, by 3½ inches deep; the introduction of a second opening 8 inches in diameter, and the reduction of the diameter of the furnace, are thus explained. Then the whole of the steam generated by these three sections was to be stored in a receiver 18 inches in diameter, and 10 feet 6 inches long; a second receiver of the same dimensions afforded perhaps scarcely sufficient additional steam room.

The Belleville boiler is of somewhat the same general type as the Howard boiler. The course of the water and steam is zig-zag, entering at one end of the horizontal tube, and passing upwards or downwards by the connecting piece at the other end. Separate upcasts and downcasts are not provided in this design, and for this reason alone it must be regarded as defective, and, if

pressed too hard, dangerous. In France the boiler has met with general favour. The use of cast-iron, which enters largely into the construction of the Belleville boiler, cannot be considered otherwise than objectionable, especially for marine purposes; because, although cheaper, this material renders the boiler heavier, and even then not wholly reliable as the shell of a steam generator.

Watt's tubulous boiler, Fig. 4, presents several excellent features, and many of the elements of danger which characterise some of the boilers already mentioned are absent. This boiler consists of a number of inclined tubes connected at either end to rectangular water chambers *a* and *b*. The tubes have a sufficient inclination from the horizontal to facilitate circulation; and the steam, after being generated in the tube and passed out of it into the higher rectangular chamber, has a free and unobstructed exit to the steam reservoir above. Again, the rectangular box *b* acts as a downcast for the water, and presents no obstruction to its flow; an excellent circulation is thus provided, and the results are said to be so favourable that a boiler which has worked on board the flat "Gertrude" for two years has had no scale or deposit, although using very impure water; and this fact is to be attributed to the scouring action of the rapid circulation set up in the water. A notable feature in this boiler is the method of connecting the tubes to the water-chambers (Fig. 5). The object in all such connections should be to obtain tight joints and ready accessibility for cleaning and repairs. In Mr. Watt's arrangement the tube is attached to the plate by a gun-metal nut screwed into the tube, and drawing it home against the plate. Access to the tubes and their connections is obtained by a door opposite the end of the tubes, the connections for fastening the doors in place being also utilised as stays to strengthen the rectangular chamber. This arrangement is simple and workmanlike, and has answered well. On the other hand, the weight of the boiler, and, indeed, probably of all tubulous boilers in their existing forms, is greater than that of an ordinary cylindrical multitubular boiler of the same power. In the course of an investigation made by the Author on this point in connection with a vessel of 1,250 indicated HP., it was found that the weight of the necessary Watt boilers, with their outer casings of sheet-iron and firebrick, exceeded the weight of ordinary marine boilers of the same power by 30 per cent. True, the pressure possible with the tubulous boiler was much greater than with the other, and this leads directly to the conclusion that, when economy of weight is of more importance than economy in consumption of coal, the ordinary

marine boiler is more suitable; indeed, there is room for much improvement in many forms of the tubulous boiler in the direction of reduced weight.

In the Perkins boiler[1] the method of forming the connection is very ingenious and highly favourable to the formation of tight joints under the highest pressures. The leading principle of Mr. Perkins' system is the continued use of the same water for periods which would be impossible if the water were liable to become charged with lubricant and decomposed matter. A special form of condenser, and a metallic alloy for use in the pistons of the engine, have therefore been employed, and, in considering the design of this boiler, it may be safely assumed that the water will remain sufficiently pure to produce no deposit, even though worked through the machinery for a much longer period than is possible where lubricants and ordinary surface condensers are employed. In describing the boiler, Mr. Perkins says, "The horizontal tubes are $2\frac{1}{4}$ inches internal and 3 inches external diameter, excepting the steam collecting tube, which is 4 inches internal and $5\frac{1}{2}$ inches external diameter. The horizontal tubes are welded up at each end $\frac{1}{2}$ inch thick, and connected by small vertical tubes, $\frac{7}{8}$ inch internal and $1\frac{5}{16}$ inch external diameter. The firebox is formed of tubes bent into a rectangular shape, placed $1\frac{3}{4}$ inch apart, and connected by numerous small vertical tubes $\frac{7}{8}$ inch internal diameter. The body of the boiler is made of a number of vertical sections, composed each of eleven tubes connected at each end by a vertical one; these sections are connected at both ends by a vertical tube to the top ring of the firebox, and and by another to the steam collecting tube. The whole of the boiler is surrounded by a double casing of thin sheet iron, filled up with vegetable black to avoid loss of heat. Every tube is separately proved by hydraulic pressure to 4,000 lbs. per square inch, and the boiler in its complete state to 2,000 lbs.; this pressure remaining for some hours without showing any signs of leakage. Experience of a very extensive character has proved that this construction of boiler can be worked safely, with great regularity, and without priming, and that the steam produced is remarkable for its freedom from moisture. The area through the vertical connecting tubes is found ample for allowing for the free escape of the steam and for the prevention of injury from overheating of the tubes in contact with the flame. Injury arising from a prolonged stoppage

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. li., p. 44; and Institution of Mechanical Engineers Proceedings, 1877, p. 117.

of the feed supply is a casualty to which all boilers are liable, but with this construction of boiler the small capacity of the sections reduces to a minimum any danger arising from such injury, and facilitates rapidity of repair."

The expression of Mr. Perkins' opinion that the area through the vertical connecting tubes is ample was probably drawn forth by criticisms to the effect that this design is identical in principle with that of the "Montana's" boilers; and that, as the latter failed from want of sufficient vertical connection between the horizontal chambers, the Perkins boiler might be expected to fail also and from the same cause. Comparing the relative proportions of vertical connection area with the capacity of the horizontal tube to be relieved in each case, it will be found that the "Montana" had two vertical necks $6\frac{1}{2}$ inches in diameter = 66·36 square inches, connected to each horizontal cylinder 15 inches in diameter and 15 feet long, or 3·6 square inches per cubic foot, and that the Perkins boiler, as illustrated, has two vertical necks $\frac{7}{8}$ inch in diameter = 1·2 square inch, connected to each horizontal cylinder $2\frac{1}{2}$ inches in diameter and 4 feet 6 inches long, or 9·6 square inches per cubic foot. In the latter case the upcast and downcast area is 2·66 times greater than in the Montana. It remains to be seen whether the satisfactory results obtained by Mr. Perkins in small examples will be continued with machinery on a larger scale. The conditions of natural circulation, carried on by separate upcasts and downcasts, can have little or no place in this boiler; but the smaller capacity of the high-pressure steam may, to some extent, compensate for this defect; although in the opinion of many experienced engineers no boiler can be safe which does not permit free and unobstructed circulation. Again, the disturbing action arising from irregular firing, or from irregular feed-water supply would, no doubt, be much augmented in a boiler having no proper provision for the regular interchange of water and steam.

The report of the Boiler Committee, in allusion to the Perkins system, says: "The Committee very much regret, in view of the further information collected by them on this subject, that difficulties should have interfered with the carrying out of their suggestion[1] as to the trial of the tubulous system; and after the further experience they have gained, it is still their opinion that the system should be fairly tested as soon as possible in sea-going ships of Her Majesty's Navy, on land, and, if thought desirable, in

[1] A recommendation that the tubulous system should be tried (1) on land; (2) on board one of the Admiralty harbour vessels; (3) on board a small sea-going man-of-war, was made by the Committee in September 1874.

steam pinnaces and cutters; boilers of the latter size being readily obtained on the plans referred to."[1] The vital feature of the Perkins system is the continued use from port to port of the same water, without leakage, and free from impurities likely to be deposited. It is admitted by Mr. Perkins that, in an engine working with a vacuum, portions of the tallow used in the stuffing-boxes of the piston rods would be sucked into the cylinder and carried through to the boiler; and beyond this it is not easy to imagine that in large engines no internal lubrication would be required; should it be necessary on long voyages to apply oil or tallow, even in small quantities, deposit on the boilers must necessarily take place; and deposit upon boilers having little natural circulation must be a serious evil. If in a boiler there is rapid circulation of water the tendency to deposit will be less, and if the construction of a boiler admits of easy access for scaling mechanically, the deposit is of less importance; but it must in fairness be pointed out that the construction of the Perkins boiler is such as greatly to hinder scaling by mechanical means. In his evidence before the committee, Mr. Perkins recommends periodical washings for removal of scale or grease. It must be evident, however, that in the case of large deposit such a system of cleaning is not likely to be satisfactory, and indeed after a Perkins boiler had been so cleaned, it would be difficult to know if deposit still remained.

In an invention of Mr. H. S. Barron, Assoc. Inst. C.E., the object is to generate high pressure steam by a comparatively slight alteration to the existing type of low-pressure boilers. Advantage is taken of the property possessed by certain oils of having a very high boiling point, and utilising them as conducting media for the transmission of heat to the water to be evaporated. By this means Mr. Barron has been enabled to design a boiler (Fig. 6) which, while involving externally no great departure from the regular low-pressure marine type, embodies the means of safely generating steam of very high pressure. The boiler consists essentially of two distinct portions: one appropriated to the combustion of the fuel consists of furnaces, flame-boxes, tubes, and smoke-box, surrounded by oil in lieu of water, the boiler being completely filled with oil; the other portion, wherein the steam is generated, consists mainly of tubular surface, presenting a rapidly augmenting sectional area for the passage upwards of the water, and ultimately of the steam. By this means a diminishing velocity is secured to the current of water

[1] *Vide* Third Report, 1877, p. 37.

passed through, and time and freedom are given for its conversion into steam. In the design shown the water in its liquid, and finally in its gaseous, condition is forced by the feed pump to traverse about ten times the oil bath in which it is immersed, and the areas of the absorbing surfaces in the successive tiers of water-tubes f, f^1, f^2, f^3, f^4, f^5, f^6, f^7, &c., being properly proportioned to that of the heating surface, a wide margin is maintained between the highest possible temperature that the oil is susceptible of acquiring and that of its boiling point, which is upwards of 600° Fahr. The water from the feed pump is delivered into the chamber e, and the successive chambers e^1, e^2, e^3, e^4, e^5, e^6, e^7, e^8, e^9, e^{10}, form the connections for the successive tiers of water-tubes. Some of the advantages which the designer believes this system to possess are: That it is safe. The deterioration of the heated surfaces ceases by reason of the employment of oil as the absorbing medium. No priming of any consequence can occur, owing to the small quantities of water within the generator, and to the prolonged stay in the oil bath which every particle of water must undergo before reaching the steam-pipe. Steam can be raised in less time than usual. The safety of the pressure-bearing parts of the boiler is independent of any particular water level. The steam is moderately superheated or dried in a vessel not exposed to the action of the flames. In some cases an efficient high-pressure marine boiler could be constructed within an old low-pressure shell, and the expense incidental to the removal of decks be thus saved. Moreover, on the score of expense it does not compare unfavourably with other high-pressure boilers, inasmuch as the entire shell, furnaces, flame-boxes, and tubes, having neither pressure to carry, nor deterioration to undergo, may be made of very thin plates, the entire weight being about the same as that of a boiler of the usual cylindrical type.

It may be objected that this boiler is designed on the principle of artificially circulated water flashed into steam, and that such a principle has never yet been satisfactorily tested; indeed, that in the case of the original Belleville boiler, for example, it was at first attempted and then abandoned. The objection is a valid one, yet the facility with which the ordinary low-pressure boiler could by this system be adapted for very high pressure, makes the invention attractive, and it seems well worth a trial.

In connection with the question of weight, which has such an important bearing upon the policy of fitting tubulous boilers for marine purposes, there are one or two points demanding consideration. When comparing the total weight of machinery of different

types, it is only fair to include the weight of coal necessary for a given number of days' consumption in each case, and, regarded in this light, the tubulous boiler by reason of its greater economy will have some advantage. Again, it is possible to construct a tubulous boiler capable of giving off the desired amount of steam for less proportionate weight than that taken in the example already referred to; that is to say, if economy of weight in connection with high pressure be desired, the grate surface may be so much enlarged in proportion to the heating surface, that abundant steam generation with small weight may be obtained, but at the cost of increased consumption. Of course this is so for all types of boilers; and it is a question of experience, so to proportion the grate and heating surfaces to each other, that the escaping gases may be of the most suitable temperature. In the best marine practice, boilers of the cylindrical type are now made with about 3 square feet of heating surface and 0·12 square foot of grate surface per indicated HP., that is, as 25 to 1. In the examples of tubulous boilers already described the proportions are approximately as follows:—

Name.	Indicated HP.	Grate Surface.	Heating Surface.	Grate Surface per Indicated HP.	Heating Surface per Indicated HP.	Proportion of Heating Surface to Grate Surface.
		Sq. feet.	Sq. feet.	Sq. feet.	Sq. feet.	
Red Rose . . .	600	72	1,587	0·120	2·64	22:1
L'Actif[1] (Belleville)	400	75	2,347	0·188	5·86	31:1
Propontis . . .	1,100	143	8,700	0·130	7·9	61:1
Montana . . .	4,500	540	21,710	0·12	4·82	40:1
Watt's	..	..	..	0·12	3·0	25:1
Perkins'	150	19	760	0·126	5·0	40:1
Barron's	..	19	494 fire-surface	..	..	26:1
			636 oil-surface	..	..	33:1

A consideration of the action of these boilers, and of the failure of some of them at sea, leads to the careful investigation of the cause which has produced imperfect working in such cases. That

[1] Engineering, March 4, 1870.

cause is not far to seek, and it may be said that nearly all the expensive experiments which have been tried and failed at sea, have failed from one common cause, viz., imperfect circulation of water. The importance of good circulation, both as regards economy and safety, cannot be exaggerated. The heat to which a plate in immediate contact with the fire is exposed may reach perhaps to upwards of 2,000°; the power of water to absorb heat is thrice that of steam, and water giving off steam will absorb, in addition, the latent heat of the steam; the heat-absorbing power of water is therefore many times greater than the heat-absorbing power of steam, and any sluggishness of circulation must, with the thin metal used in tubulous boilers, be a cause of danger. Again, the smaller body of water contained in the water-tube boiler produces a less gradual, and therefore less safe circulation than in the fire-tube boiler, and the increased friction due to higher pressures is a further disadvantage against which the former has to contend. Some cases of imperfect action have been caused by the mistaken employment of water-tube boilers for low pressures. The water-tube boiler is, from its very nature, adapted only to high pressures, because a difficult circulation is in one respect assisted by a high pressure; as the cubic capacity of 1 lb. of steam varies inversely as its pressure, a given boiler will evaporate no greater weight of steam at 10 lbs. pressure than at 1,000, although the bulk will be greater; now circulation is the mutual changing places of the steam generated and the water about to be flashed into steam, and the space to be traversed in this interchange of place will vary inversely as the pressure, and the smaller the space to be traversed the more easy the circulation. It would appear then that to employ a tubulous boiler for other than very high pressures would be a dangerous mistake. It is difficult to say how far these opposite results of high-pressure steam, viz., increased friction and decreased volume to be circulated, compensate for each other, but the balance of evidence appears to show that higher pressure on the whole facilitates the circulation in a well-designed tubulous boiler. In well-designed boilers of the common type there is no obstruction to the vertical flow of the steam from the surface upon which it is generated to the steam space above the water level, but in most examples of water-tube boilers the steam must travel horizontally before it begins to ascend.

An interesting discussion has recently taken place in the pages of a scientific journal upon the comparative advantages of vertical and horizontal heating surfaces, especially as applied to high-pressure boilers. The question appears to lie within very narrow

limits. It is necessary to remember that the action of circulation is due to gravity alone, the steam bubbles tending to rise and the water to fall, and, where evaporating water is confined in a narrow tube, if that tube be placed in an exactly horizontal position the steam bubbles can only rise through the height of the diameter of the tube, and will accumulate at the upper side; if they do eventually have any circulation it will be borrowed from the currents ascending through a vertical, or approximately vertical, line; and the velocity of the circulation will be correspondingly reduced. On the other hand a vertical tube, unless properly supplied with water led through a separate downcast, may have no real circulation, but may be full of steam struggling upwards against water struggling downwards, and destroying circulation. It appears then that the employment of tubes in horizontal, or nearly horizontal, positions will not give good results; but the use of tubes of properly proportioned diameter and length, in a position sufficiently inclined to allow the steam bubbles to ascend along the tube by gravity, and having afterwards an uninterrupted flow to the steam chest, will be safe and efficient. Under all circumstances a boiler should possess separate upcasts for the newly generated steam, and separate downcasts for the water to take its place. A point of equal importance is the free access to all parts of the boiler for the purpose of cleaning. From the nature of the case the tubulous boiler is difficult to arrange in this respect, and some examples, after working well for a time, have failed from the large deposit of scale for which no ready means of removal were provided.

In concluding this Paper it is desired to revert to the largeness and difficulty of the question. It is, as yet, by no means exhaustively understood; but the foregoing facts and theories will at least serve to show that many valuable lessons may be deduced from the experiments already made, and that there now is a fair basis on which to carry out further investigations. The Admiralty Committee has, so far, undertaken the solution of the boiler problem, and it may be that that Board will see fit to prolong to their fullest conclusion the labours of those gentlemen who have so ably and so patiently examined the difficulties of this important question.

The Paper is illustrated by several diagrams, from which Plate 6 and the wood-cut Fig. 1 have been compiled.

[Mr. FLANNERY

Mr. FLANNERY said Fig. 1 (p. 126) showed graphically that economy did not increase nearly in the same ratio as pressure. The horizontal line at the bottom indicated the pressure along its length, and the curved line the economy in its distance from the horizontal line. It would be seen that the curve became more nearly parallel to the horizontal line as the pressure increased. Diagrams of the "Propontis," "Montana," Perkins, and Circuit Boilers were also shown for the purposes of the discussion.

Mr. E. A. COWPER regretted that the substance of the Paper did not carry out the title, and that no definite construction of boiler had been pointed out as being good, and "adapted for very high pressures;" also that no absolute pressure of steam had been advocated as the right pressure to use. It would appear that the Author intended to treat on the subject of marine boilers; and it was quite necessary to keep the consideration of land and marine boilers separate, as they did not follow the same rules as to weight, power, bulk, floor-space, cost, or even chimney draught. If it was intended to give a history of tubulous boilers and their failures, such well-known boilers as "Spiller's," "Hancock's," "Shand's," "Goldsworthy Gurney's," and others ought to have been described. "Spiller's," for instance, was a very successful boiler, having the water in the tubes, crossing the inside of the fire-box, the tubes being 3 or 3½ inches in diameter, of moderate length, and at an inclination of 3 inches in the foot. It enabled a higher pressure to be carried than had hitherto been used afloat, and worked better than most old boilers of the time, though it had given place to the modern marine boiler, with the fires in long furnaces and return tubes over the furnaces. This boiler with a strong cylindrical shell, and well stayed ends, was an admirable one for marine purposes. The Author had not recommended any one tubulous boiler as a good, practical and safe marine boiler, but had stated that some of the boilers were capable of being modified so as to make them successful. Mr. Cowper thought tubulous boilers that required sheet-iron casings lined with brickwork had better not be sent to sea, for fear they might be shaken to pieces. Tubulous boilers weighing above 30 per cent. more than ordinary marine boilers should not be put on board ship unless great advantages could be proved to be obtainable. Tubulous boilers that primed so seriously on the slightest provocation as to burn out quickly were not safe in ships. Boilers with a mixture of long tubes, some 2½ inches and others 12 inches in diameter, in which the steam and water were so confined that the tubes were burnt out and exploded again and again, were to be noted only

for the purpose of avoiding them, as any approach to such incongruous constructions must lead to serious disappointment and loss. It was not a question of adding to this or to that boiler an extra outside return tube to admit water for the heat to act upon, nor of increasing the size of certain narrow necks to confined bottles of water exposed to fire; but it was the question of constructing a boiler without such steam traps and confined spaces, or entailing the necessity of using pure water, that should commend itself to the notice of engineers. He was not one of those who thought that perfection had been attained either in boilers or in engines, nor did he agree with the Author that engines had been attended to to the neglect of boilers; for about one hundred and seventy patents for boilers were taken out yearly, and a great deal of thought and ingenuity had been expended on them, though much labour had been misapplied owing to a want of appreciation of the necessities of the case. This was proved by the drawings exhibited, and by the results of the working of such tubulous boilers. The proper function of a marine boiler was to make a large quantity of steam on a limited area of floor. To work well, it was necessary that the water should have room to allow the bubbles of steam formed on the metal to rise freely. The steam ought not to be kept down by a horizontal covering of metal; it should be allowed free egress from the water, otherwise priming and burning of the boiler would take place, from the steam driving the water before it, thus leaving the metal dry. Horizontal, or nearly horizontal, tubes exposed to a hot fire were particularly objectionable in both these respects, as the water did not give a passage to the steam, and the steam being produced in large volumes, filled the tubes, and drove the water before it, leaving the tubes dry, and there being no medium to keep down the temperature of the tubes, they were quickly burnt or worn out. In some tubulous boilers it was impossible to tell to a foot where the level of the water was; and he had known cases where the stoker did not know whether the water was in the glass gauge or out of it. Such boilers were troublesome to manage and to feed with accuracy. A very hot fire playing at right angles against a plate dominated the temperature of the plate, even if there were plenty of water behind it, and would soon burn the plate. This fact was well known in reference to the use of the waste heat from puddling furnaces. A thin wall of brickwork was commonly put against the plate to receive the impact of the flame, and then when the current of heat had been turned around the boiler, there was no further occasion for a protection of brickwork.

He had often seen these effects, and had known other boilers properly protected work well.

In the management of boilers one point should never be lost sight of, and that was to keep the boiler as nearly as possible at an even temperature. More injury was done to boilers from variations in temperature than was commonly admitted; thus, if the bottom of a boiler was cold whilst the top was hot, very great strains would be brought upon it, particularly if it were a long one. Cases had occurred with double flued Cornish or Lancashire boilers, in which the bottom of the boiler was so cold as to admit of being handled, whilst there was a pressure of steam of 20 lbs. per square inch at the top. A few Galloway tubes were often put in the flues, partly to promote active circulation, which was so important. With regard to the degree of advantage to be obtained theoretically by the use of steam expanded down to a little above a fair vacuum, he did not quite agree with the results as given on the diagram. There was no difficulty in setting out the true expansion curve, as de Pambour's experiments showed the bulk of the steam at given pressures in comparison with the bulk of the water that had produced it; and as the latent and sensible heat together were practically a constant quantity for steam at different pressures, the proportionate bulk of steam at once gave the ordinates of the expansion curve. He had thus set out the true expansion curve in 1849, and his late brother, Mr. Charles Cowper, had then published it.[1] He had proved the truth of the curve again and again. The economy to be effected theoretically could be calculated best from the above expansion curve. By using steam at 125 lbs. per square inch total pressure as against 70 lbs. total pressure, it would be found that an economy of only 21½ per cent. could be gained. This he considered was a poor recompense for the additional trouble, expense and annoyance in reference to joints, valves, pistons, &c., due to using steam of the higher pressure; and if there had to be added a bad boiler, or one difficult to work regularly, he did not think the change would be economical. No practical result had been put before the meeting to justify such a change, as the result obtained for a short time with the "Propontis," viz., that of 1½ lb. of coal per HP., with a pressure of steam of 150 lbs. per square inch, had been far exceeded with a pressure of 55 lbs. of steam from ordinary strong marine boilers.

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[1] *Vide* Institution of Mechanical Engineers Proceedings, 1877, p. 147, and plate 47.

Mr. LONGRIDGE agreed with a great deal of what Mr. Cowper had said, particularly in his conclusions with regard to the advantages of very high-pressure steam. The Author seemed to assume, as a fact admitting of little dispute, that further economy in steam work was to be obtained by the use of very high pressures. So far as the theoretical curve went there was a certain advantage; it did not increase so rapidly when the pressure was increased to a great point; but practically it was well known that the advantages that might be supposed to follow did not accrue, either from the hyperbolical curve or from the real adiabatic curve. Various statements had been made to explain this: that the cylinders absorbed so much heat; that the heat flowed into the condenser, as if it were some independent fluid that ran away wherever it could get, and the like; but he had never seen anything which satisfied him as to the real cause. Again, supposing it to be an advantage to employ a very high pressure, say 200 lbs. or 300 lbs. per square inch, the Author said there were no means, with existing boilers, to obtain it. With a modification of the locomotive type any pressure could be obtained that could be got by the extraordinary boilers described by the Author; 200 or 250 lbs. could be obtained with the greatest safety. As regarded space, weight, and price, a modified form of the locomotive boiler would beat in every respect the boilers referred to. It might be said that in the marine engine there was no steam blast; but nothing was easier than to substitute a bellows or fan blast, which would succeed perfectly; the amount of force required was very small, and combustion could be obtained to any extent desired. The Author had given no details of the cost of the boilers, their bulk, weight, and evaporative effect; and hence he failed to see any great value in the Paper. Certainly, a good description was given of five or six new types of boilers, all of which, however, seemed excessively complicated, and none of which he should like to use. It was stated that in the "Propontis" boiler 1 HP. was obtained from $1\frac{1}{2}$ lb. of coal. One indicated HP. was a very indefinite unit; but allowing all the advantage possible, and supposing that every lb. of coal evaporated 13 lbs. of water, that would give 20 lbs. of water represented by 1 HP. The "Propontis" had an 1,100 HP. boiler; consequently it evaporated 22,000 lbs. of water per hour. The heating surface was 8,700 square feet, and every foot evaporated $\frac{1}{25}$ cubic foot of water per hour. In a recent Paper[1] he had given the results of the evaporation from nineteen locomo-

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. lii., p. 101.

tives taken indiscriminately, and the average of those was $\frac{1}{3}$ cubic foot evaporated from every square foot of surface; so that the "Propontis" boiler did only a third or a fourth of what an ordinary locomotive boiler could do with great safety and ease. He had no means of checking the other boilers described, but he felt sure, from the proportions given in the table, that the same remarks would apply to them. The advantages claimed for them seemed to him perfectly illusory, and none of them could compensate for the disadvantages in regard to cost, bulk, weight, and economical effect. The Author had stated (p. 129) that "the tendency of these layers of gas, which have parted with their heat, to continue in contact with the plate is very great, and . . . . there is comparatively little opportunity for those layers of gas furthest from the plate, and which have not yet cooled down, to change place and part with their heat in the most direct manner, unless some means of breaking up the current of the flames are interposed." If there were any facts bearing out that opinion he should be glad to hear them. It was a favourite idea of the late Mr. Charles Wye Williams to put things inside the tubes, whirl them round, and so create currents of air; but the results he expected were never obtained. It was, besides, quite contrary to Mr. Longridge's opinion as to the distribution of heat under those circumstances. He believed the heat was radiant, and passed not from particle to particle by conduction; but that it went, like all radiant heat, with enormous velocity from the centre to the circumference. The Author further stated: "The balance of evidence appears to show that higher pressure on the whole facilitates the circulation in a well-designed tubulous boiler." Perhaps he would state where that evidence was to be found. He agreed that there was nothing more important than good circulation. In the Paper to which he had referred there was a constant *m*, the value of which he had assumed to be 7, but he believed by an increased circulation that constant might be nearly doubled. Sir John Leslie, many years ago, showed, with regard to a body cooling in air, that taking the rate of cooling in still air to be unity, by swinging the body round at 60 feet per second the rate of cooling would be increased tenfold. The same law applied to water, and therefore any means of increasing the circulation in boilers would be productive of great advantage.

Mr. Rich wished to call attention to the loss of economy in high pressure boilers in consequence of the greater heat of the gases escaping up the chimney. In any high-pressure boiler the steam, and also the bulk of the water, must necessarily be at a much

higher temperature than in a low-pressure boiler. In a boiler under 500 lbs. pressure per square inch the temperature of the steam and of the bulk of the water would be nearly 500°; and instead of the gases escaping in the chimney at a temperature of 350°, as in an average good boiler under 60 lbs. steam, they would escape at 550° to 600°. That extra heat wasted up the chimney was one of the elements which reduced the economy of high pressure boilers.

Mr. SPENCER said, as he was about to put engines and Perkins boilers of 800 HP. in a ship now building, he might be permitted to make a few remarks with reference to the use of boilers on the tubulous principle. He might observe that it was quite impossible to get anything but suggestive Papers on such a subject, as engineers were unwilling to bring their failures into notice. Suggestive Papers, however, were serviceable in bringing out opinions, and perhaps answered better than Papers advocating some particular plans by those whose motives might be suspected, which could certainly not be the case in regard to the present Paper. If the only advantage to be derived was the theoretical gain in economy from doubling the pressure of steam, not much could be urged in favour of very high pressures; but this was only one element among many. It might seem an anomaly, but his opinion was that an increase of pressure was the only road to safety. Many existing defects arose from too low a pressure, and as soon as it was found necessary to use a pressure much beyond that employed in the dangerous boilers now at work, a new era of safety would be arrived at, and a class of boilers would be obtained which would excite no fear of bursting. The great thing to avoid in all steam boilers was the liberation of a large body of water from a large chamber. By reducing the quantity of water to such an extent that in case of the boiler bursting it would simply cause a little vapour to be thrown into a large space without doing injury, a great advance would be made. In very high pressures, it was necessary to use small chambers, and that permitted the use of comparatively thin material. With the existing large boilers it was often necessary to use 1 or 1¼ inch plate, and the fire inside and the cold atmosphere outside gave rise to all sorts of contractions and other injuries that destroyed the boiler in a short time. In a Perkins boiler the weakest part required about 12,000 or 13,000 lbs. to the square inch as a bursting pressure; taking the ordinary strength of 20 tons per square inch, there was a large margin to begin with; and therefore if by any accident the feed was irregular,

the boiler could be severely strained without danger. He had come to the conclusion that excessive circulation in boilers never occurred except where there was some distortion in the boiler which produced the circulation. A double-ended marine boiler, for instance, with narrow water spaces, and large heating power, was turned into an ejector with a very active circulation, and great destruction to the engine. In the case of a common pot put over the fire what circulation was there? Steam was thrown off at the top, but unless the pot was forced beyond a certain amount the steam would be comparatively dry. The Perkins' boiler was always in a constant state of priming, and yet dryer steam it was impossible to get. He could not explain it, but there was no circulation as the term was generally understood. He was satisfied that any accident like that in the "Montana" was impossible. If the "Montana" had had a better arrangement at the top of the boiler, giving a more regular pressure on the different sections, the accident probably would never have happened. He could not see that there was any possibility of the Perkins boiler going wrong at sea, unless it was allowed to remain red-hot for a long period. He was putting boilers into a yacht of 700 tons, with full sail power, under circumstances most favourable for trying a new system. He should never have thought of introducing the system had he not great faith in it. He believed that the use of very high pressure steam, say 300 or 400 lbs. per square inch, was the only road to safety in marine steam engineering. By some such system as Perkins', with small chambers of moderate thicknesses, and a large surface divided into sections, nine-tenths of the present troubles with steam-boilers on board a man-of-war would be avoided. The small tubular system, however, could not be adopted except on some such plan as Mr. Perkins proposed, with a supply of pure water, a difficulty not so great as it appeared. No internal lubrication could be used with the Perkins system, but the pistons were self-lubricating. To ensure durability, tight joints and pure water were indispensable. It had lately been discovered in France that it was easy to separate the impurities coming from surface condensers before they passed into the feed pipe. That was another step in the right direction.

Mr. SCOTT RUSSELL remarked that the Paper directed the attention of practical men to a critical point in the history of the steam-engine, and especially the marine engine. He agreed with Mr. Spencer that the present state of marine boilers was extremely

dangerous. He also agreed that safety for the future was only to be secured by going in for very high pressure, with all its advantages, and taking precautions against its dangers. With high pressure, a boiler could be used of a given power, of much less bulk, and of much less weight. The dangers attending it were priming and scale. With regard to circulation in boilers, he not only did not believe in its value, but he thought it was one of the great elements of danger. It should be given up altogether, and the water converted into steam at one effort. Making two moves was only a confession that a bad boiler had been produced, so that after the water had been sent through the whole circuit it was unable to give off its steam, and had to be sent round again. The cold water should be lifted from the bottom of the boiler, and gradually warmed, so that when it reached the top it could take the shape of steam. The vertical tubulous boiler, well proportioned and contrived, would do everything that was wanted. Taking a single vertical tube (he did not care if it was sloped a little), if it were properly proportioned in surface and volume, and placed judiciously, so that the right quantity of heat came into it to convert the water into steam by the time it reached the top, that would be the best, simplest, strongest, and most durable boiler that could be made. He recommended the tubulous high-pressure boiler as the boiler of the future. He had no crotchets of his own, although perhaps he was the oldest boiler-maker present. Younger men were now doing the work, and the great value of such a Paper as that under discussion was that it would give a right direction to their efforts. No doubt there were dangers to be avoided. The danger of scaling could be overcome in many ways, into the details of which he could not enter. There was a great difficulty in having a proper furnace for a tubulous boiler. How it was to be overcome he need hardly say, but an envelope made of bricks or similar material was out of the question. With such additions a simple boiler would be converted into a complex one, and a safe and durable boiler into an unsafe and short-lived one. A great blunder was often committed in putting the coldest part of the heating surface where the greatest heat was wanted, and the hottest part where the least heat was wanted — a wasteful and injurious arrangement. Yet many persons thought they were making a good boiler and furnace when they put the cold water, just as it was injected, at the hottest part of the fire. A good boiler should have the coldest part where the water was coldest, and the hottest part where the

water had arrived at the final stage and was ready to flash into steam. He wished to direct attention to another mode of making a thin metal boiler fit to receive high pressure. He referred to the stay system—three sets of stays at right angles with one another—first introduced by himself in 1827. He took a cubical furnace, and put parallel stays very close to each other in one dimension, another set close to each other in the second dimension, and a third set in the third dimension. He made boilers for marine and land purposes, of various sizes, of a thickness of only $\frac{1}{8}$th inch, and they worked with a pressure of ten atmospheres. The plan was afterwards adopted by Mr. Stephenson in the fire-box of his locomotive engine. For a long time a great firm in Liverpool did not believe in the square fire-box, and for twenty years they sent out cylindrical ones. By adopting the cubical boiler he had described, a square furnace could be used to hold the fire, all surrounded by water, and parallel tubes in any direction, very much like the modern marine boiler, except that the latter had not been carried out to the extent he had mentioned. He did not recommend this boiler as better than the tubulous boiler, although, as an invention of his own, he had a great affection for it. He regarded Mr. Perkins' tubulous boiler as a grand discovery. It required however, for its successful working, wise people to make it, and well-trained engineers and stokers to work it.

Mr. Spencer said he had omitted to mention, with regard to the weight of tubulous boilers, that in the case of the four boilers equal to 800 indicated HP., to which he had referred, there was a saving in boilers and water of 40 tons. There was no brickwork of any kind, and the boilers could be handled with impunity.

Mr. McFarlane Gray said that the Author had very clearly explained the present position of the question of boiler construction for higher pressures. His examples very well illustrated the principal directions in which boiler schemers were moving; and if there were little to commend in the plans described, the Author was not responsible for these plans. He had produced a Paper which contained ample material for discussion. Evidently there had been a great deal of scheming and altering, and but little, if any, progress. It appeared to him that the mere alteration of form or arrangement of heating surface, unless to serve some purpose of convenience or cheapness of construction, was of no importance; it did not matter much how the heating surface went so long as there was enough of it. He did not agree with those who condemned the present form of cylindrical marine boilers for the

highest pressures now in use. He thought the facility they afforded for thorough internal inspection was apt to be undervalued. He considered that with such facilities there could be no excuse for such a boiler bursting. He had for many years been asserting that there was practically nothing to be gained by higher pressures unless under exceptional circumstances. He considered 100 lbs. pressure as rather above than under the highest economical pressure. A table given by the Author as a quotation from him (Mr. Gray) had been constructed on the occasion of the publication of a Paper by Mr. Alfred Holt in 1873. The following statement occurred in that Paper: "It may safely be asserted that if suitable boilers could be constructed for still higher pressures, and an engine with durable parts devised, the present consumption might be halved to-morrow."[1] It was to meet this remark that the table of twos was put forward. The principle of the table was Mariotte's law. The modifications due to the less percentage of back pressure loss at higher pressures, and the increase of initial energy, and the decrease of pressure by volume during expansion according to thermodynamic expansion, were set off as cancelling each other. These items therefore had not been neglected; they were eliminated. The table by the Author, in which back pressure was fully calculated, but the thermodynamic reduction neglected, was fairly corroborative of the simpler table of twos. It was a significant criticism of the value of higher pressure economies, that in his recent Paper before the Institution, Mr. Holt gave the same figures for the progress in economy which he had given in his pamphlet of 1873; but he now considered that the greatest possible advance to be ever attained with the steam engine at any pressure could not exceed 20 per cent. upon the present efficiency. This statement was a revise of Mr. Holt's forecast, after five years' further experience, and those who were now so sanguine in their expectations from higher pressures, ought to weigh well together these two statements. The starting data of the table of twos were from Mr. Holt's Paper, and the arrangement alone was his own. In reference to the oil and water boiler described in the Paper, a plan was now being introduced for the prevention of priming in boilers by the use of petroleum with the water. He thought these circumstances might be taken as suggestive of a boiler to work with water and oil more successfully than the compound oil and water boiler of Mr. Barron. As water and petroleum

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[1] *Vide* Remarks on the construction and strength of the cylindrical parts of steam boilers, by Alfred Holt, p. 4. 8vo. Liverpool, 1873.

were not mutually soluble, he should expect that the pressure of the mixed vapours of water and petroleum would be the sum of the pressures of the vapours of each at the temperature of the liquids. The difference between the temperature of the gases in the flues and that of the liquid in the boiler would therefore be greater than in a steam boiler at the same pressure, and the efficiency of the heating surface would be thereby increased. In the plan of Mr. Barron the opposite of this would occur, and more than twice the present heating surface would be requisite. Then, in respect to priming, it was not known how petroleum operated to prevent priming; but in the case now considered, where the excess of pressure due to petroleum was to be considerable, it was easy to see how it might act with that result. Priming was an inconvenient excess of steam globules in the liquid. If the two liquids did not mix, the vapour globules formed in the liquid, if any, would each contain the vapour of only one of the liquids. But the vapour pressure in a globule must be at least equal to the pressure of the liquid about it; and it also could at most be only the pressure due to the temperature of the liquid; that was, it could not as vapour exceed the pressure due to one of the liquids, while the pressure to which it was subjected was the sum of the two pressures. It followed that no vapour globule could be formed in the liquid, and therefore there could be no priming. The vapour would be in that case formed at the liquid-steam surface, and the solid matter left would be suspended in the liquid. When the conditions were such as permitted the formation of vapour globules in the liquid, these would be mostly formed at the metal-liquid surfaces, and the solid matter would be left as scale on the metal. These remarks were merely the communication of the suggestion which occurred to him while reading the Paper, and if it were followed out to the complete explanation of the action of petroleum in boilers, or to the better understanding of priming, the expression of this thought here might, in his opinion, not be quite valueless. In the use of petroleum in boilers with surface condensers, it would probably be found that the oil would, at each evaporation, become more and more volatile, until its molecules were as light as they could be. This successive evaporation was called in America "cracking" the oil. If this were the case a light ought not to be brought in contact with the gas that would be found in the hotwell. With regard to air hanging about the surface of the tubes he thought there must be some mistake, considering the enormous velocity with which

the products of combustion passed out of the chimney. With reference to the trouble occasioned by cylindrical boilers leaking at the bottom, that had been considerably amended by cementing with a cement mixed with pitch, which made it adhesive and elastic.

Mr. Crampton said he was sorry to find the question of fuel consumed by the engine mixed up with the boiler question. The two things should be kept perfectly distinct. One made the steam, and the other used it. Then the Paper gave no amount of surface for the quantity of water evaporated in a given time, or the temperature of the gases escaping into the chimney—most essential points in the investigation of the boiler question. His opinion was that it mattered little what kind of surface was employed; almost any kind or condition of surface sufficed if the flues were arranged accordingly. It had been stated that there was no circulation in the Perkins boiler. He could not conceive that steam could escape from a particular surface, and that water could gain access to that surface, without circulation. The extraordinary results produced by the Perkins boiler had astonished him greatly. He could not see how the circulation took place, but that it did take place was certain. No doubt it was a good thing to get a boiler to stand a very high pressure, even if not used; but it was doubtful whether high-pressure boilers were of any commercial value up to a certain point. Occasionally there were reports of great results by the indicator diagram—200 or 300 lbs. pressure of steam per square inch and 1½ lb. of water consumed per indicated HP. He thought these reports were fallacious. It was necessary to know what was the actual power developed by a given weight of steam, and the quantity of water evaporated by a given surface in the boiler before coming to a decision as to what was best. The diagram gave the quantity of power absorbed in the engine, but it did not give the work done. He had made some experiments upon the question about twelve years ago, being always opposed to excessively high pressures, and not believing in the value of very great expansions. The curve showed there was not really the value that might be expected, particularly at very high expansions. Taking 40 lbs. steam-pressure the value was 21; at 80 lbs., 25; showing a theoretical gain of 20 per cent. From 40 lbs. to 300 lbs. the gain was 100 per cent.; but then the problem was how to deal with 300 lbs. pressure of steam as against 40 lbs.; and all the parts would have to be strong in proportion. In the experiment to which he alluded, and which had been continuously at work for twelve years, he had the opportunity of doing as he pleased. There were water

tanks which could be filled with water, and he proportioned the engine (20-inch cylinder and 20-inch stroke) up to 80 lbs. pressure of steam, in order that he might reduce from 80 lbs. down to whatever might be required to do the work. He tried experiments at 80 lbs., 60 lbs., 40 lbs., and 30 lbs., varying the expansion from 5 to 12 times. The result in all cases showed, by the theoretical diagrams, as nearly as possible the theoretical gain to which he had referred; but when he looked into the tank, the corresponding quantity of water was not there. That led him to inquire how it was. He ought to have lifted 20 per cent. more water with the same fuel; but practically he did not get any more. Where were the 20 per cent. lost? He believed that the additional friction by extra condensation and various other causes consequent upon high pressure would absorb the 20 per cent. and that it was better to use 40 lbs. instead of 80 lbs. pressure of steam, particularly in small engines. In very large engines where the losses would be relatively less some little advantage might be obtained, but not much. He had not used a drop of oil in the engine. The tops and bottoms of the cylinders and the sides were well jacketed, and the whole apparatus was well clothed. There was sufficient condensation of the steam (only super-heating from the temperature of the boiler in the jackets) to give enough water in the cylinders to lubricate them. There was surface condensation also; he had therefore no oil in the boiler. He thought it a pity that some established facts were not obtained to show what was practically the ultimate useful pressure attainable. His impression was that the high pressures sought for would not be found commercially advantageous.

Mr. R. W. Perkins remarked, with reference to the development of steam in proportion to the space occupied by the boiler and the weight, that a great deal depended upon the intensity of heat and rapidity of combustion, but a mechanical difficulty had hitherto presented itself. He had found on one or two occasions that in thus intensifying the heat to obtain the highest development of steam, the grate bars melted in three or four hours. To avoid this he adopted the plan of a bar in connection with a steam jet or other suitable blower, which system he had patented, and he thus obtained steam in the proportion of two boilers to three, and the bars were cool at the end of the operation. It would allow of anthracite coal being used, which, from the slowness of its combustion, had hitherto been found quite inappreciable as a steam generating fuel except in cases where this was compensated for by

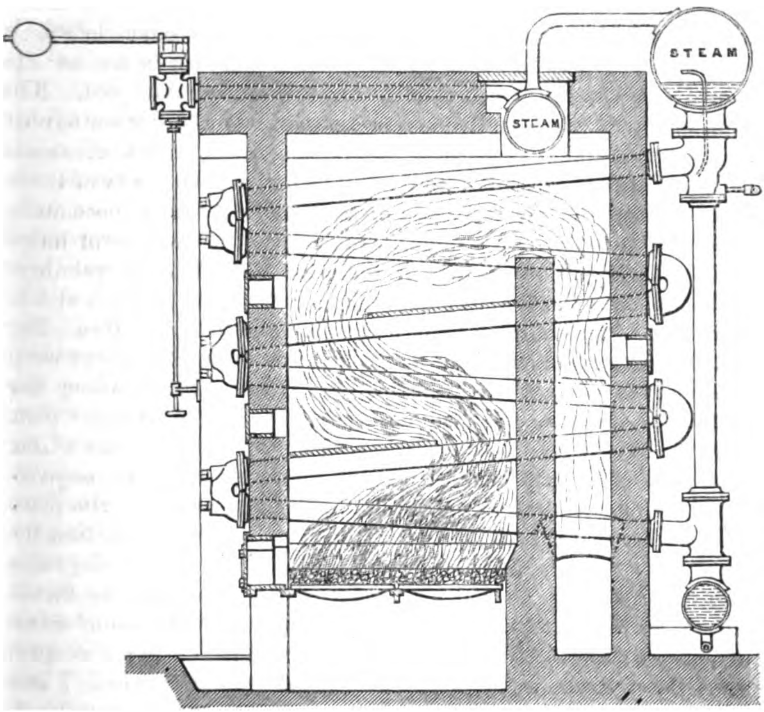
a very large boiler capacity, and therefore increased area of heating surface. Naval officers would be delighted to use anthracite coal, because it would enable a ship at sea to escape the observation of the enemy. The bars were hollow, and perforated so that the blast went through them, and then into the fire through the holes in the side. The intensity of the heat would be in proportion to the cooling of the bars, because both were determined by the rapidity of the cool air going through the holes.

Mr. Olrick said he agreed with the Author that the great point to be considered in the construction of boilers was to make sure of a good circulation, so as to convey the more heated water away and allow the cooler portion to come under the influence of the heat as soon as possible. The failures of tubulous boilers had all arisen from the want of circulation, and from the boilers not being accessible to examination and removal of incrustation. He did not think that a proper circulation could be said to exist in a boiler where the upward current was not absolutely separated from the downward current. The upward current was a constant natural self-acting rising current, of a mixture of steam bubbles and hot water, contiguous to the heating surface, and which helped to wash off the steam bubbles clinging to the heating surface and to carry them through the water level to the steam space above; whereas the downward current consisted of water only, and as it was absolutely separated from the upward current, it was impossible that it could obstruct its course. The result was that in boilers with good circulation there was a great economy of fuel. By putting a Galloway tube into a Cornish boiler there was a separation of the two currents, and as the steam and water in the Galloway tube was considerably lighter than the water outside, between the shell of the boiler and the flue, a perfectly natural self-acting current would take place. The reason why the Davey Paxman boiler was a good evaporator was that there was a separation of the two currents. In the Field tube it was the same. Taking away the internal circulation, what would be found? The same state of affairs as in a railway station where there was only one staircase, and one set of people trying to leave the station and another set trying to catch a train at the same moment. The result would be an absolute block of passengers. Mr. John Fowler, Past President, had exercised a wise discretion in making at all his railway stations two staircases, thus compelling a separation between the two currents of passengers, one representing the up-draught, the other the down-draught. The best of the

boilers described by the Author was the Watt, because there was a separation of the up and down currents; still a certain amount of the down-draught tube, acting as heating surface, and some ebullition took place that ought not to occur, because the down-draught tube ought not to be used as heating surface. It had also a number of flat surfaces that ought to be avoided in the case of very high pressures. The Root boiler consisted of a great number of small boilers, each tube being a separate boiler, and there was no circulation in the tubes in the sense he had described. The Belleville boiler had been applied to a number of steamers in the French Navy, but it had suffered from the same defect of circulation. The Perkins boiler, which was the same as the "Montana," had been successful while the other had not. The "Montana" dealt with a steam pressure of 60 or 70 lbs. per square inch, and at that pressure the number of cubic feet of steam evaporated from 1 cubic foot of water was 300; whereas the Perkins boiler dealt with pressures of 500 lbs. per square inch, and at that pressure the number of cubic feet of steam evaporated from 1 cubic foot of water was only 60; so that there was three times the area for the upward current to pass through, and five times less cubic contents to pass through the vertical tubes. The Author had expressed a doubt whether a down-draught, independent of the up-draught, could be made. Mr. Olrick had placed on the wall a representation of what he had called the "Circuit" boiler, Fig. 2. It had been designed to remedy the shortcomings of the Belleville boiler, which had parallel tubes of the same diameter throughout; and the steam had to find its way to the steam dome in an irregular manner, as there were no means for separating the upward and downward currents. In the Circuit boiler, the tubes were all slanting to assist the ascent of the water and the steam. It would be observed that, as the tubes got higher and higher, they were larger and larger in diameter, to allow for the space that the steam bubbles mixed with water would occupy. There was a down-draught tube which enabled the water to pass downwards, and the steam went into a dome at the top of the boiler, with a super-heater or dryer. Every joint in the boiler was outside the fire. The whole boiler was made of wrought iron. To examine the boiler for dirt it was only necessary to take out a small plug in the first instance, and if required the joints of the bend could then be broken. The feed-water was put in at the top of the down-draught tube, so as to enable the cold water that came in to be heated to a sufficient temperature to precipitate

any impurities that might be in the water, and leave them in the receiver at the bottom of the boiler, from whence they were blown off by the usual blow-off cock, thus preventing the impurities from being carried in over the heating surface. The back or down-draught tube was not heating surface, therefore the water was cooler than in the other parts of the boiler. If the boiler were applied to a man-of-war, and a shot were fired into the engine-room, the explosion would be far less destructive than in

FIG. 2.



The Circuit Boiler.

one of the existing boilers 10 feet in diameter. Again, if the boiler were entirely destroyed, it would be only necessary to send to the store-keeper of the ships, and in less than a week a new boiler could be built up without breaking a single plank. Considering how much depended upon the fleet for defence and attack, he thought that the Government would be spending money well in trying the Watt boiler, the Perkins boiler, and the Circuit boiler.

Mr. FLANNERY said Mr. Cowper had charged him with omissions in the Paper, because he had not described certain boilers used twenty years ago; and Mr. Longridge had also complained of the omission of details; but what length did those gentlemen expect the Paper to run to? Mr. Cowper had further complained of the title, and had asked what he meant by "very high pressures"? The title had been suggested by the Council, and he had simply adopted the suggestion. He had not, as Mr. Cowper seemed to imply, advocated all the boilers he had described as being a solution of the boiler difficulty, or as the proper types of boiler to be used, for he had seriously condemned many of them. The curve in Fig. 1 (page 126) involved a slight error, being calculated by hyperbolic logarithms, and not being strictly an adiabatic curve. This had been stated in the Paper. With reference to the contention of Mr. Longridge, that locomotive boilers possessed all the advantages of boilers such as those described in the Paper, and not many of their disadvantages, he would dispose of it by saying that a locomotive boiler had necessarily a confined steam space. In the case of large cylinders with a slow movement, there was a tendency to priming. In locomotives there were small cylinders working at high speed, and steam was withdrawn from the steam space almost continuously. Mr. Longridge had asked for an explanation of the statement with reference to contact with heated gas when passing along the surface of the flue. He had no other authority to produce than that of Mr. Wye Williams, who had made the statement and supported it by experiment. It seemed reasonable to suppose that if the heated gases were passing along the surface of the plate at a high speed they would not convey their heat to it so directly as if they impinged upon it at right angles. Mr. Longridge also inquired why he had assumed that high pressure promoted circulation? Circulation was the mutual changing places of water and steam. With high-pressure steam there was a given weight, say 1 lb. of steam occupying space inversely as its pressure; and it was evident if the space occupied was smaller, the space through which it must travel, in order to change places with water, was also smaller. It was, therefore, easier to circulate water and steam in a boiler of high pressure than in one of lower pressure. It would be found, from a table of temperatures, that there was a very slight difference in the sensible temperature of steam at high pressure and of steam at low pressure. There might be many times the pressure of steam with very little increase in the sensible temperature. With reference to the alleged danger

and weight of firebrick, the calculation referred to in the Paper as to the Watt boiler, included a large amount of firebrick, which was necessarily a considerable item in the total weight of that boiler. Mr. Perkins had succeeded in doing away with this objectionable feature, because he had introduced, to contain the flame, a double sheet of iron plate with charcoal between. This prevented the radiation of heat, and it was much lighter than firebrick. Mr. Crampton had charged him with not dealing with the subject completely, because he had mixed up the questions of the engine and the boiler, and had spoken of the consumption of coal in the engine and boiler, and not of the evaporation of water. But Mr. Bramwell had shown again and again how important it was to make sure not to mistake evaporation for priming—not to assume that the water primed into the engine was evaporated. Unfortunately, there were few experiments with water-tube boilers on any large and reliable scale showing their evaporative efficiency; and he was unable to bring them forward. All that he could do was to give the results shown in the "Propontis" and other examples where tests had been made of the consumption of coal. With reference to the remarks of Mr. R. W. Perkins he might mention that he had spent a fortnight or three weeks in conducting experiments made on that gentleman's system of fire-bars. His object seemed to be to reduce the space and weight necessary for a given development of steam. Those experiments gave the very highest encouragement to believe that two boilers would suffice where three were now required, because of the more active combustion, due to the blast passing through and afterwards out of the bars. He was glad to find that an engineer of so much experience as Mr. Olrick agreed with the views expressed in the Paper. It appeared that there were two sorts of boiler; one was a water-tube boiler which might be successful, in which there was ordinary circulation, or, as Mr. Olrick had put it, an upward and downward current perfectly separate; in the other, which might also be successful, as suggested by Mr. Scott Russell, the water rose without change of motion from the lowest part of the boiler to the highest, and then evaporated. In that case there was no circulation in the ordinary sense of the word.

Mr. R. V. J. Knight remarked, through the Secretary, that higher pressures and greater expansion would entail increased condensation in the cylinders, which condensation would, he

feared, take away at least one-half of the gain promised by theory, and would, in fact, practically limit the extent to which expansion might be carried with economy. As the present type of high-pressure marine boiler would continue in use for the next few years, it might be useful to see how far it could carry higher pressures with safety. He would confine his remarks to the strength of the cylindrical shell, the part to which the Author more particularly alluded. Fortunately, reliable steel plates could now be obtained, which would be of great assistance, unless boiler constructors were too much fettered with official restrictions. Before stating the inferences he had drawn from experiments made at his request, he would record the results obtained, as they might induce other engineers to publish some of the valuable experiments which no doubt they had undertaken. To ascertain the practical damage done by punching holes in steel plates for boilers, and the effect produced by connecting, the following preliminary experiments were made. From a plate of Landore steel $\frac{5}{8}$ inch thick, three plates, 6 inches wide and 16 inches long, were cut. In the first and second plates, three 1-inch holes were punched, at 3 inches pitch, arranged zig-zag. The first plate was not annealed after it had been punched, but three red-hot rivets were closed in the holes in the usual way, and it was then planed down to 3 inches wide, so that two of the rivets were cut through their centres, and one rivet was left in the centre of the plate untouched. The second plate was made red hot after it had been punched, and was then treated in the same way as the first plate. In each plate were therefore two sections of equal area, but both plates broke through the centre rivet-hole. The third plate was planed only 2 inches wide, having the same section as the first and second plate, and was left in other respects in the state in which it was received from the maker. The first plate broke under $31\frac{7}{8}$ tons pressure, the second under $32\frac{5}{8}$ tons, and the third under $33\frac{1}{8}$ tons, or under $25\frac{1}{2}$ tons, 26 tons, and $26\frac{1}{2}$ tons per square inch of net area. The loss was 2 per cent. for the plate annealed after punching, and 4 per cent. for the plate not annealed. As in practice all punched holes are closed with red-hot rivets, it was considered only fair to give them any benefit which might be derived from the rivets by supporting or annealing. Instead of continuing this class of experiments, it was thought more useful to test double-riveted lap joints of steel against the same kind of joint of iron. Two joints of best Staffordshire iron, and three joints of Landore steel, were prepared,

all alike, except in thickness, which was proportioned inversely to the expected ultimate tensile strengths of iron and steel. All the joints were 9 inches wide, all the holes were punched, and each joint was connected by six 1-inch iron rivets. The iron plates were $\frac{7}{8}$ inch thick, and the steel plates $\frac{5}{8}$ inch thick. The iron joints were torn through the rivet-holes by $71\frac{1}{10}$ tons, and $73\frac{2}{10}$ tons pressure on a mean tensile strength of $13\frac{8}{10}$ tons per square inch of net area of joint. The ultimate tensile strength of the iron plates was $21\frac{1}{2}$ tons per square inch. The first steel joint was not annealed after punching; the second steel joint was warmed to a blue heat after punching; the third steel joint was made red hot after punching. In each case the iron rivets were sheared by $93\frac{1}{2}$ tons, $91\frac{2}{10}$ tons, and 92 tons pressure respectively, or $18\frac{1}{2}$ tons per square inch of rivet area. The ultimate strength of the plates was 27 tons per square inch. The plates sustained, without fracture, a tensile strain of $24\frac{6}{10}$ tons per square inch of net area of joint, or about 10 per cent. less than the ultimate tensile strength, although two of the joints had not been annealed after punching. The $\frac{5}{8}$-inch steel joints were 27 per cent. stronger than the $\frac{7}{8}$-inch iron joints, but per square inch of net area they were 78 per cent. stronger; therefore for lap-jointed boiler shells it would have a great advantage over iron, but for but-jointed boiler shells the advantage would be only that due to the relative strengths of iron and steel, say 30 per cent. The die used in all cases was one-sixth of the thickness of the plate larger than the punch, which proportion had been found to work well both for iron and steel of mild quality. Applying the term "Theoretical strength of the joint" to the product of the net area and ultimate tensile strength of the material, then the actual strength of the joints (including those published in Vol. li. of the Minutes of Proceedings, page 131), expressed in percentages of the theoretical strength, would be as follows:—

	Per Cent.
For the 1-inch lap joints	56
„ „ $\frac{7}{8}$-inch lap joints	64
„ „ 1-inch but joints	90
„ „ $\frac{5}{8}$-inch steel lap joints . . .	90

In all his experiments on double-riveted lap joints with punched holes, he found this percentage increased as the thickness of the plates decreased, being about 85 per cent. for iron plates $\frac{3}{8}$ inch thick. Although these iron joints varied in strength from 56 up to 90 per cent., yet he knew of no published rules which

took these great differences into account. The Board of Trade rules only allowed 5 per cent. increase of strength for the but over the lap. Supposing three boilers to be of the same diameter, the first made with a double-riveted lap-jointed shell 1 inch thick, certified by the Board of Trade for 70 lbs. pressure, then if the second boiler was made with a double-riveted but-jointed shell 1 inch thick, it would be equally safe for 112 lbs. pressure; and if the third boiler was made with a double-riveted but-jointed shell, 1 inch thick of steel, it would be equally safe for 146 lbs. pressure. The working pressure allowed by the Board of Trade would be about 75 lbs. for the second, and about 95 lbs. for the third boiler. Although 146 lbs. pressure could not be called a "very high pressure," yet it would be welcomed by all advocates of higher pressures. His own impression was, that although higher pressures would lessen the consumption of fuel, yet that the additional economy effected in the working expenses would be very small.

Mr. E. B. Marten observed, through the Secretary, that the Author was evidently unaware of a report on some most careful experiments on the evaporative duty of five samples of tubulous boilers conducted by the American Institute at New York, of which the following was an abstract:—

"Some of the most careful experiments on record, where the evaporative duty was ascertained by condensing the steam and measuring the actual heat conveyed, gives the duty of the Root boiler as 8·76 lbs. of water per lb. of coal.

"The same experiments showed the duty of other tube boilers as nearly the same, viz.:—

	lbs.
The Allen	8·76
The Phlegor	8·70

and compared them with two others with external shells—

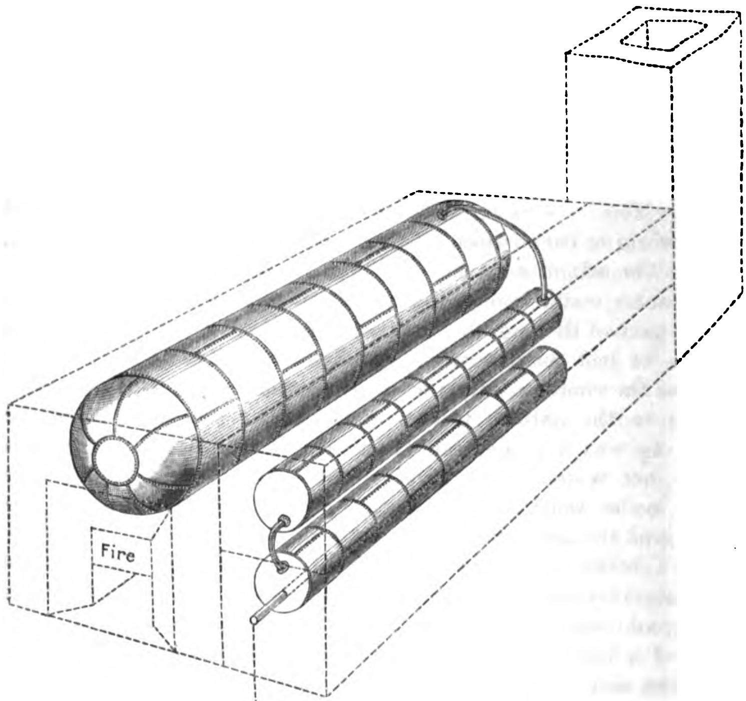
	lbs.
The Lowe	8·55
The Blanchard	9·41."

One of the advantages incident to the system of periodical inspection of boilers which he had conducted for many years, was, that the same set of observers noted during long periods the behaviour of many thousands of boilers of every variety of type, so that a larger amount of useful information was gained than would be possible in a private practice. From his own observation and experiment he could confirm the Author's statement that nearly all

boilers, placed in proper circumstances, would give about the same evaporative duty; but in practice, economy of capital, space, and time, was often of more importance than mere economy of fuel. He had watched the working of the Howard, the Root, and the Perkins boilers, like those mentioned by the Author; and also other tubulous or small water-space boilers, such as the Benson and the Harrison. Some of them had shown that tubulous boilers did not always mean freedom from explosion and loss of life; but further attention to details of construction would reduce that danger to a minimum. The chief cause of difficulty was the need of pure water, and of ready control over the fire, to enable it to be reduced as suddenly as the demand for steam. The result was that unless the very high pressure for which tubulous boilers were intended was made use of, many of the old forms of boilers were better. The Perkins boiler alone fulfilled the expectations formed of it, owing to the unusual care bestowed on its mechanical details, and to the adoption of the important principle of working the same water continuously, making the water as it were a reciprocating part of the engine. By means of small glass eye-pieces in boilers he had made many observations on circulation, and had come to the conclusion that a steady advance of the water from the coldest to the hottest place was the better principle, the water going one way and the heat the other. Circulation, or the return of the hot water to the cooler part, involved loss, because the whole boiler must thus be kept up nearly to the heat of the steam, and the gases must escape at a high temperature. A simple form of boiler was shown in Fig. 3. It was very effective and economical because circulation was impossible, although there might be a good deal of useful agitation just over the fire. It consisted of a large plain cylinder under which was the fire. There were two side tubes of the same length as the larger one, but of half the diameter; and the heat passed completely round the upper, and then returned around the lower tube. The only connections were the single copper pipes, and these not being rigid each cylinder was free to expand. The water entered where the heat was least, at the back of the lower side tube which was quite full, and then rose into the upper side tube, also full, and fell into the large cylinder where the last increment of heat was added to evaporate it. If the water contained scale-making impurity it was left in the first tube, where, the heat not being sufficient to harden it into scale, it was blown out as mud. Evidence of the correctness of this principle had been shown in the fact that since cross tubes

had been added to the old slow-working Cornish and Lancashire boilers, more work had been done in a given time, but at a loss of evaporative duty from the fuel. This had only been partially prevented by the economiser, to bring the heat back from the flues beyond the boiler, which acted the same part as the side tubes in the boiler described above, with more danger of being choked with

FIG. 3.



Boiler without circulation, the feed advancing steadily in the opposite direction to the heat, so that the whole available heat is brought back from the flue. Feed-pipe carried to the back. Boiler also capable of very high pressure if two smaller cylinders are substituted for the large evaporating one.

scale. It was estimated that about 75 per cent. of the stationary work of the kingdom was done by the Lancashire or Cornish boilers of various types, and it was natural that the general belief should be that such ponderous and complicated structures were alone suitable for the purpose. It was to be hoped that investigation would be continued; and that a more handy evaporator would be the result, capable of varying the supply of steam as needed, so

as to do away with the present necessity of a large but dangerous store of force in the shape of highly-heated water.

Mr. D. Phillips remarked, through the Secretary, that he did not think many engineers of practical experience held the same views as the Author. As to the gain to be effected by much higher pressures of steam than those commonly in use, what seemed to him to be wanted was knowledge acquired from practical illustrations, and on a much larger scale than hitherto attempted, and not assumptions derived from theory, or deduced from mathematical problems. In estimating the gain which it was believed by many would be effected by pressures five or six times greater than that in ordinary use, on a large scale, and for long sea voyages, numerous points might be considered. Among these were: first, the original cost of the machinery; secondly, complexity of the machinery, and the wear and tear of the principal working parts, especially if it should be attempted to derive the full benefit of expansion with too few cylinders; and, lastly, the much greater care which would have to be exercised by, and the responsibility which would necessarily devolve on, those in charge of the engine-room. Difficulties, no doubt, presented themselves when the leap was made to construct and work marine engines with double the pressures carried sixteen or seventeen years ago; but those difficulties had, happily, been overcome, leading to a great saving, especially in fuel. How far this might be successfully carried out in practice was doubtful. Of the various plans of steam generators to withstand high pressures exhibited by the Author, there was but one capable of carrying a higher pressure than 50 lbs. per square inch with safety, and that only so long as the conditions of working were safe. For 500 lbs. pressure and more, Perkins' water-tube boiler was the only example he knew of capable of being worked with safety; and, as one of the many examples brought to the notice of the late Boiler Committee, he could speak of it as being fully up to the mark. No doubt the larger the diameter of the tubes in a generator of this type the greater must be the cubical contents; and that being so, the greater would be the probable mischief in case of an explosion. Again, the smaller the diameter of the tubes the less accessible they must be for examination *in situ*, which might be objected to in such a boiler as the Perkins; but, on the other hand, if the boilers could be worked with water free from solid matter, as pointed out in the report of the Boiler Committee, and which was effected by the Perkins plan on a

small scale, no danger need be apprehended from incrustation and consequent "burning" of the iron. Should, however, sea water, though not on account of the salt or grease from the engine, reach such a boiler to a serious extent, no doubt it would in a short time, as in the "Propontis" boilers, prove fatal to it. That danger was to be apprehended at sea, especially with large engines and boilers running on long voyages. However perfect the machinery, hitches sometimes occurred, and the tallow-kettle, or oil-can, would be resorted to. Again, the best condensers might become leaky, and sea water find its way freely into the boilers. The area of the vertical connecting tubes of the Perkins boiler was ample, as pointed out by the Author, "for the free escape of the steam, and for the prevention of injury from overheating of the tubes in contact with the flame," in the rather small boilers at work; but he should doubt their efficiency, if boilers of this type were to be made on a large scale, with a proportion of heating surface corresponding to their size, especially if made much higher. In this respect, and as a steam generator, the boiler might be easily modified, and, he thought, with great advantage.

On the vexed question of different waters, and their effects on the iron boilers, on many important points chemists were not only at variance with each other in theory, but at variance when the results of their laboratory investigations were compared with those obtained by actual practice. The practical engineer had been misled, in many instances, by reports founded on such results; but more so, perhaps, by some of the theories well known to the chemical world, and only superficially by engineers. Now it was generally accepted as a fact, that distilled water *per se* was, under almost all conditions, much more injurious to iron than rain or ordinary fresh water, especially if used "over and over again;" and that the denser or more salt the water became from evaporation, the more destructive it was to iron. On these points some interesting experiments had been carried out by the late Boiler Committee, and the results, as yet unpublished, had proved the fallacy of many existing theories. These experiments showed that, in the comparative absence of air, there was scarcely any oxidation of the surfaces, after a period of two years, with either of the following waters, viz., salt water $\frac{3}{32}$, and sea water $\frac{1}{32}$ in density, well water, rain water, or water distilled from sea water. In similar waters, with air present, but without changing the waters, except a little in one or two instances on account of leakages, the results were very different,

both in the nature and severity of the corrosion. The tubes which had fresh water in them on the whole suffered more seriously than those which contained salt water, the corrosion in the former being of a local character; deep pits being confined principally to the neighbourhood of the water level, and extending but diminishing in depth and size to the bottom of the tubes. The corrosion in those tubes which contained salt water was confined to the steam space, principally near the level of the water, and gradually decreasing towards the top of the tubes; but it was more general, though of less depth, than when fresh water was employed. With regard to the action of water distilled from sea water, under the same conditions of working, there was scarcely any difference between either the nature or the extent of the corrosion, compared with the action of fresh water, except that it was very slightly more marked at the lower part of the steam span in the boiler containing distilled water. With a short series of plates, 4 inches long and broad, and $\frac{3}{8}$ inch thick, having bright surfaces and suspended in open vessels at a temperature of about 210° or 212°; the first was in a vessel containing soft fresh water; the second in a similar vessel with water distilled from the same water as the first; the third was in a vessel with an admixture of both fresh and distilled water; and the fourth and last, in a boiler working at a pressure of about 50 lbs. per square inch, fed with water from the latter vessel. The following results were obtained after a trial of thirteen months' duration:—

			Grains.		Grains per Square foot of Surface.
First plate	in water supplied direct from the main	lost	264·0	or	1006·5
Second „	„ distilled from the same water	„	313·5	„	1462·0
Third „	in an admixture of both waters . .	„	405·0	„	1544·0
Fourth „	in boiler	„	20·0	„	76·2

The difference in the nature of the corrosion was remarkable; the first sample was corroded in the form of pitting (somewhat like shot marks); the second was corroded uniformly, nearly smooth all over, with the edges quite sharp; while the third was corroded unevenly, but more irregularly in form than the first; and the fourth specimen was scarcely affected. In this experiment an incident occurred which no doubt had a great deal to do with the result. The grease (mineral oil), resulting from the use of a surface condenser for six months and a half out of eight, got into the second vessel in rather large quantities, and still more so into the third, and protected the plates in a most extraordinary manner.

Thus, during the first period of the experiment, while the first plate lost 180 grains, the second only lost 13 grains, the third 11 grains, and the fourth 7 grains. From this it would appear that the first and fourth plates could hardly have been subjected to the influence of the oil during the six and a half months that the condenser was at work, although some of the oil had reached the boiler. When this was found out, the vessels and boiler were thoroughly cleansed of oil, and the communication with the surface condenser was closed for the rest of the time. He would next refer to one of a series of experiments carried out in glass bottles containing water at atmospheric temperature, with round pieces of polished iron of the same size, connected by a small brass rod to a brass disc, the other conditions being the same in each case, and the time twelve months.

	Grains.		Grains per Square foot of Surface.
First sample in water of $\frac{12}{32}$ density, or nearly saturated, the loss was only	5·6	or	105·1
Second sample in water of $\frac{6}{32}$ density, or 6 times that of the sea, the loss was only	6·1	„	114·5
Third sample in water of $\frac{3}{32}$ density, or 3 times that of the sea, the loss was only	12·7	„	238·4
Fourth sample in water of $3\frac{1}{2}$ density (sea water), the loss was only	13·2	„	247·8
Fifth sample in water, as used at Whitehall, the loss was only	16·2	„	304·1
Sixth sample distilled from fresh water, the loss was only	12·0	„	225·2
Seventh sample distilled from sea water, the loss was only	12·4	„	232·0

In another series, where the area of the surface of the water was a little larger, and the quantity of water in the bottle one-third less, the following results were obtained:—

			Grains.		Grains per Square foot of Surface.
First sample in distilled water,	*per se*,	the loss was	34·4	or	535·8
Second „ „ „	from sea water	„	23·2	„	359·9
Third „ in sea water		„	24·0	„	372·5

In this case the difference was greatly in favour of sea water, the time being the same in the first and third cases (twenty and a half months); but in the second instance, only a little more than twelve months. The proportion of loss between the first and second cases therefore was as 39·6 in water distilled from sea water to 34·4 in the distilled water *per se*. In the first series the loss was less in the distilled than in the sea water, which was due to the absence of salts when brass was connected to the iron, although the corrosion showed no appearance of "galvanic

action," as it was even and uniform all over. He would refer, next, to an experiment to ascertain the comparative effect of waters in marine boilers, one boiler being fed from an ordinary jet condenser, and the other from a surface condenser. The boilers were filled, and the waste was made up with sea water, and care was taken to reduce the waste as much as possible, the same water being worked "over and over again" in the latter case. In the former case the water was changed entirely fourteen times, and in the latter only twice during a period of nearly thirteen months. The time of work was about one-third of the whole; but with the boilers always under banked fires, except when the machinery was being overhauled. Although both iron and steel plates were used, he would only refer to the loss in the iron plates, which were bright, 10 inches long, 8 inches wide, and $\frac{7}{16}$ inch thick.

	Grains.		Grains per Square foot of Surface.
The average loss in the two plates suspended in the boiler fed from the jet-condenser was	1203·1	or	1018·2
In the two plates in the boiler fed from the surface-condenser the loss was only.	820·3	„	687·3

This indicated a saving of about 48 per cent. in favour of the surface condenser, and changing the water as little, and opening the boilers as seldom, as possible. To show still more clearly the effect of changing the water, he would mention two cases obtained from another series of experiments, with engines having surface condensers in both instances, and in vessels in H.M. Navy. In one case "blowing off" and changing the water in the boilers were carried out to a most extraordinary degree, and in the other as before. In this, as in the above experiment, he would only refer to the iron plates, which were two in number, 4 inches long and broad by $\frac{3}{8}$ inch thick in each case. In the former plate the average loss was 556 grains against 118·2 grains in the latter, whilst the time occupied in the latter case was a little more than three times that of the former. This showed that at this rate of deterioration the boilers under the latter conditions would last about fifteen times as long as those under the former, provided the plates were of the same thickness, &c. The pressure carried in the first case was 40 lbs., against 30 lbs. to the square inch in the second; but he did not think this would make any practical difference. The last subject to which he would refer, bearing on the deterioration of marine boilers,

was the "fatty acid" theory—a theory which scarcely ever escaped discussion when the question of deterioration of boilers was brought forward. He would refer to two experiments on this point; the first was carried out in a tube of copper, and the second in one of iron, the condition of working being the same in each. Water of $\frac{2}{32}$ density, $\frac{1}{2}$ lb. of tallow, representing the lubricant, and air, were admitted weekly, under an average pressure of 38 lbs. to the square inch. Iron placed in a vessel under similar conditions only suffered to the extent of 65·3 per cent. more than the iron tubes, which were each 8 feet long by $2\frac{1}{8}$ inches in diameter internally, with the water occupying about two-thirds of their length. In another tube under the same condition, except that 2 oz. of copper filings were added, the general surface of the tube, especially at water level and upwards, was in far better condition than of those tubes which contained neither tallow nor copper filings. Moreover, the plug in the bottom of this tube was the least affected of any in a group of ten tubes. Now if copper, in a metallic form, placed in a vessel of this kind, had no effect, on either the tube or plug, during two years, what effect could it have on surfaces of a working boiler? Scarcely any. He thought from the few experiments alluded to, and from some concluded by the late Committee, and others in progress, that the causes of the corrosion in marine boilers would be found not to be due to working them with "too pure water," or water "worked over and over again;" nor to the copper carried over in metallic particles, or in a dissolved or oxydised state; nor to water distilled from sea water; nor to fatty acids; although some of them might contribute, slightly, under certain conditions of working, to destroy the iron of the boiler. He believed the deterioration to be principally, if not entirely, due to the free admission of air and carbonic acid, resulting from the constant change of the water by "blowing off," &c., and from air mechanically introduced by the air and feed pumps; and last, but not least, to the exposure, which boilers underwent, to atmospheric influences when idle and open. Although it was impossible to work engines without slight leakages, and the use of pumps, yet the waste resulting from the former should be made up, and the water from the condenser should be delivered into the boilers charged with as little air as possible. This was, he believed, the secret of the satisfactory condition, internally, of Perkins' boilers. Moreover, these boilers, from there being no necessity to clean them, except occasionally, could be kept free from air when idle,

in fact under a vacuum; and on account of the limited area of the stop valves and other connections, the boilers were less liable than ordinary ones to become leaky, either from friction of the steam, or from wear and tear of the surfaces. Of this he could vouch, from having watched the operation of disconnecting a section of the factory boiler of Messrs. Perkins, after it had been idle about three weeks.

Mr. F. J. Rowan observed, through the Secretary, that he thought the Author was in accord with the majority of engineers in his remarks on the advantage to be derived from the use of steam of higher pressures than those now in general use in steam ships. To what the Author had said about the diminished value of the areas for very high pressures enclosed in the hyperbolic curve, he would add that some further arguments, derived from the increase of strengths necessary, and the difficulty of lubrication, &c., had been advanced by Professor Osborne Reynolds among others, to show that beyond about 200 lbs. per square inch, the ratio of advantage derivable was much diminished. He fully concurred, also, in the Author's remark with reference to a suitable construction of boilers, that the requirements of this high pressure led directly to reduced diameters, or more correctly to reduced areas for fracture, and thus to water-tube or tubulous boilers. Expedients for strengthening existing cylindrical boilers, such as that recently proposed by Dr. Siemens, while they reflected credit upon the skill of their inventors, were from their exceptional character more likely to involve serious risk than the more simple method referred to by the Author. The increase of strength derivable from reduction of water and steam spaces was of course the direct argument in favour of such reduction, and here he would observe that while the Author acknowledged this, he betrayed inconsistency in his approval of the design of the Watt boiler, of which an essential part was a rectangular box with large flat surfaces at each end of the tubes. He was aware that these flat surfaces were stayed to obtain the required strength; but this did not eliminate the defect in design to which he alluded.

The subdivision of boilers necessarily resulted in an increase of the surface exposed to the action of the hot gases from the furnaces, and it followed that a larger portion of the heat produced by combustion might thus be utilised. It was in connection with this part of the subject that the Author, in common with many others, had introduced the opinion that, in order to realise this economy, the heating surface must be disposed horizontally so as

to present surfaces at right angles to the currents of the heated gases; and as the gases usually ascended directly to the funnel, the only conclusion to be drawn was that the boiler must consist of horizontal tubes. He ventured to combat this opinion on the ground that none of the requirements of the problem, when closely examined, demanded such conditions. All the reasoning to be met with on this subject as to the comparative value of various kinds of surface, and as to the behaviour of layers of gas, proceeded on the assumption that the gases were allowed to escape either by a gradually ascending current or by direct vertical movement. But the conditions were altered if the gases were compelled to take a downward course before being allowed to escape upwards to the funnel. This effectually disposed of "layers" of gas, because it compelled thorough intermixture and circulation of the gases, the colder portions being those which of necessity escaped first. This result was independent of all question of position of heating surface. In fact, considered merely as affected by the hot gases, all kinds of surface would have an equal value if exposed to a body of heated gases treated in this way. Evidently, too, by this method of working, a nearer approach was made to a position of having the power of maintaining the contact between the hot gases and the boiler surface for the maximum time necessary to abstract the heat. It had been carried out in practice in almost all the examples of Rowan and Horton's boilers made since their introduction in 1858, and had been an important element in their economy of fuel. With chimney or suction draught, however, it was impossible to possess to the full the power of maintaining contact between the gases and the surface, until the gases were cooled down to the lowest point, because a certain portion of heat must in such case be sacrificed to the necessity for the creation of draught, in order that combustion might be assisted. This defect might be removed by introducing forced combustion, or combustion under pressure, and the speed of travel of the gases might with proper appliances on this system be regulated at will.

He differed from the Author about the position and powers of boiler surfaces and the action of layers of gas, because Professor Clerk Maxwell's investigations of the theory of heat, and those of other workers in the same field, had shown that the transmission of heat from the gases to the water was regulated simply by the time of contact of the heating medium with the iron of the boiler, and by the proper circulation of the water by which one face of the metal was maintained at a lower temperature than the other.

The rate of flow of heat was steady and could be measured, as it was proportional to the area exposed to heat, to the time and to the difference of the temperatures of the two faces, and inversely proportional to the thickness of the metal. The circulation of the water in the boiler, in this view of the case, very properly assumed a position of great importance, and he was glad to find some remarks by the Author on this part of the subject with which he heartily concurred. Yet, in opposition to opinions which he had expressed in other pages, he maintained that a properly constructed water-tube boiler could not have urged against it that it presented difficulty to the obtaining of good circulation; but that, on the contrary, it offered conditions for the realisation of thorough circulation such as could not be obtained in boilers of the ordinary cylindrical form or of other designs. If only natural modes of action were observed and provided for, the escape of the steam and upward movement of the currents of heated water, inducing the regular supply, from below, of portions to be heated would proceed in perfect order, and all the more efficiently that small quantities were set in motion in the individual parts of the boiler. The mere rapidity of circulation involved no danger, provided there was circulation, and not merely the rapid formation of steam without a proper supply of water. In boilers composed of horizontal, or nearly horizontal, tubes, there was great danger of the existence of this latter state of affairs, and the more rapid the formation of steam in them, the greater would be the obstacle to the access of water to the heated surfaces. When, however, as in a boiler of vertical or nearly vertical tubes, the escape of the steam itself assisted the movement of the water, it was manifest that rapid movement was no defect. It was really a decided advantage where there was any danger of a deposit of solid substances from the water; and a very interesting proof of this and of the comparative value for circulation of vertical and horizontal water tubes was in the possession of Mr. Parker of Lloyd's. He had shown him some tubes cut out of the same boiler, where, in the same part, some were placed horizontally and some vertically; the horizontal ones were salted up, some solid and some nearly so, while the vertical ones had merely a thin coating of scale.

He was surprised at the difference between the Author's remark, that "the escape of steam will be facilitated by the disposition of the heating surface horizontally," and that where he urged as a defect in most water-tube boilers that "the steam must travel horizontally before it begins to ascend." He must

also challenge the fairness of his summing up of the recent discussion on circulation, in which he took some part, for the Author had instituted a comparison between horizontal tubes and vertical tubes which had not a proper supply of water to their lower end, and had concluded in favour of inclined tubes, as if vertical tubes did not exist in other conditions. If inclined tubes were good, as the Author indicated, what was the value of vertical or nearly vertical tubes properly supplied with water as in the boilers which he advocated?

If fresh water only was used in generating steam, there need be little or no provision made for access to the interior surfaces for cleaning. This removed one objection to water-tube boilers; but in this case corrosion from the action of carbonic acid and oxygen introduced into the boiler with the condensed steam must be provided against. There was also, in boilers working in long-voyage steamers, the chance of the leakage of a small quantity of sea-water into them with the feed, and this would intensify the corrosive action. From these actions no boilers were safe, and those of Mr. Perkins were subject to the same risk as the rest, so that it was necessary to introduce some protection. The Boiler Committee had shown that zinc afforded some protective power. In addition to its use there seemed to be two methods of working, viz., that of adding chemicals to the water to decompose some of the salts and render it alkaline, as proposed by the Committee; and that of protecting the boiler surfaces by the formation on them of a thin insoluble scale from fresh water, as proposed by him in a Paper on "Boiler Corrosion." The Author had referred to the "Propontis," but in his Paper which he had quoted, from the Transactions of the Institution of Naval Architects, he dealt with this example of Rowan and Horton's boiler as if it had been the only one tried. This was not the case. In the discussion on Mr. Holt's Paper,[1] he alluded to the fact that in one shape or another these boilers had been worked from the year 1858. During that period the s.s. "Actif," of the French Navy, quoted by the Author, was one of the instances, she having been originally fitted with these boilers, and even after their removal retaining the three cylinder engines, which in 1874 (according to a report from M. Sabattier, the Minister of Marine) were working with a Belleville boiler. Several sets of boilers had been started, both before and since, on the same plan as that of the boilers of the "Propontis," but without the defect which

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. li., pp. 110–113.

ruined them; and all were working well to this day. This, as also the investigations conducted by Mr. Samson, of the Board of Trade, with the "Propontis" boilers, was proof that the boilers of the "Propontis" failed through a defect in construction. The absence of communication between the steam spaces was a very grave defect in these boilers, and led both to priming and to the obstruction of circulation, which resulted in the chambers over the fires being frequently without water. Unfortunately, the gravity of these things was not understood until damage had been done.

The weight of the boilers of the "Propontis" was greater than that of other examples of this system, because the makers chose to make the entire front of the casing of heavy cast-iron plates. Although the weight of these boilers, as ordinarily arranged for merchant steamers, was a little more than of cylindrical boilers, yet they could be arranged to weigh no more; and in any case the water contained in them was considerably less, so that the total weight was no greater than in ordinary practice. It was a mistake to say that the firebrick tiles, which were bolted to the inside of the sheet-iron casing, and which formed its lining, worked loose at sea. In all the examples of these boilers such a result had never been experienced, and any individual pieces which were accidentally broken were easily replaced.

The new design, which he had patented, was not proposed to meet defects in previous plans, because these were working successfully; but in order to provide an arrangement suited to the small height available under the shot-proof decks in vessels of the Navy, and where low space was for other reasons all that could be had. The arrangement of forced combustion was more than a mechanical stoker, as this was generally understood, and it enabled him to do without an ashpit under each boiler, besides affording the advantages in combustion to which he had referred.

With the Author's concluding paragraph he was sure that many who were interested in this subject would cordially agree. Apart from the importance of the subject considered scientifically, the fact that a sum amounting to from £100,000 to £200,000 was spent annually on boilers in the Navy alone was enough to show that the interests involved in it were large. He ventured the suggestion that a committee to investigate and report upon designs, as well as for the supervision of boilers, would be of great public service, and would be a valuable auxiliary to the constructive department of the Admiralty.

Mr. Flannery, in reply to observations communicated in

writing through the Secretary, subsequent to the reading of the Paper, remarked, through the same medium, that, although some of these contributions were important, they did not demand a lengthy reply. He would first observe, however, that Mr. Cowper had been disappointed that the Author had omitted to advocate some special designs of boiler working at a definite range of very high pressure; and that he had confined the Paper principally to the consideration of marine boilers. So experienced an engineer as Mr. Cowper might have been expected, while joining in the discussion, to have supplied the deficiencies of which he had complained; but, beyond a passing allusion to Spiller's boiler, he had not done this. The statements made by him to the effect that boilers of 30 per cent. above the average weight should be proved to have great compensating advantages before being used at sea; that boilers that primed seriously on the slightest provocation were "not safe in ships;" that great injury was done to boilers from unequal temperature at top and bottom, and other statements of the same kind, could not be regarded as novel or valuable contributions to the discussion. Mr. Cowper suggested that, although only trifling alterations might have been necessary to make certain designs of tubulous boilers safe, yet the proper remedy was the construction of a tubular boiler without confined spaces. It was, however, difficult to see how this construction was possible. His opinion that engines had not been attended to with more success than boilers would, no doubt, be changed upon an inspection of the engines at the Paris Exhibition. There refinement had been added to refinement to produce economy by expansion in the cylinder; while the fact of the Admiralty re-appointing a Boiler Committee showed that much yet remained to be understood as to the construction of boilers. The experiments upon steel joints, detailed upon the authority of Mr. Knight, must be regarded as a valuable contribution; and, taken in connection with the data given by Mr. Parker at a meeting of the Institution of Naval Architects,[1] might be said to show nearly all that could be learned by experiment as to the comparative strength of butted and lapped steel joints. The results obtained by Mr. Parker, especially as to the increased percentage of loss from the punching of steel plates, as the thickness increased, were confirmed by the experiments which Mr. Knight had now made public. Mr. Marten's remarks upon the economy to be gained by having the

[1] *Vide* Transactions of the Institution of Naval Architects, vol. xviii., p. 334.

coolest water close to the coolest gases, and the hottest water and the steam close to the hottest gases, were very pertinent to the subject. His arrangement was exceedingly ingenious, and seemed likely to extract as much heat from the gases as was possible, consistently with the proper amount of chimney draught. It did not appear, however, that this arrangement was essentially different from that of a powerful feed-water heater attached to the boiler. Unless great care was taken that steam was not generated in the smaller, or feed heater, portions of the boiler, they would be dangerous. By judicious arrangements of the flues, as shown in Fig. 3, this objection might be met; but there must still remain considerable circulation of the water in the larger or main portion of the boiler. The experiments related by Mr. Phillips on the question of corrosion in high-pressure boilers might be regarded, when coming from that gentleman, as a proof that, upon the question of longevity, Mr. Perkins was adopting the proper construction; and that, if it were proved that in large machinery lubrication could be dispensed with by his system, he had solved the question as to the preservation of high-pressure boilers. Mr. F. J. Rowan had charged the Author with inconsistency, because, while advocating a cylindrical form as the best for strength, he had also expressed approval of the Watt boiler with its rectangular boxes at the ends of the tubes. In this, however, there was no inconsistency, because, although the rectangular form, even when stayed, was not the most suitable for strength, yet, when other considerations necessitated its adoption, it might properly be used. In the Watt boiler, the advantages of the rectangular box forming a chamber common to the horizontal tubes, and allowing easy access to them, was so great, that its use was perfectly justified, provided due care was taken in the arrangement of the stays. There could be no doubt that Mr. Rowan's arrangement to regulate the current of the heated gases was capable of producing the best result by making them strike at an angle against the water tube. He had not suggested that the use of vertical tubes was incompatible with a proper arrangement in this respect. The specimens of water tubes alluded to as being in the possession of Mr. Parker could not be said to prove that horizontal tubes were more liable to deposit than vertical tubes, unless it were shown that the arrangement in the boiler was such as to cause the extra deposit in the horizontal tubes to be due to the horizontal position, and to that alone. If, for example, two horizontal water tubes were connected by several vertical tubes, it was to be expected that the deposit of scale would be greatest in the lower horizontal tube; but this was

entirely due to the position of the horizontal tube in relation to the others. If it were one of a set of horizontal tubes, acted upon by a current, the tendency to deposit would not be so great. The question of weight was of the greatest importance in connection with marine boilers. Mr. Rowan was wrong in assuming that the Author's statement as to comparative weight did not include the water in each case. In three instances that had come under his notice he had found the weight of the tubulous boiler, with its water and fire-brick casing, to be excessive; but the use of sheet iron and charcoal casing would no doubt much reduce the weight.

May 21, 1878.

WILLIAM HENRY BARLOW, F.R.S., Vice-President, in the Chair.

No. 1,574.—"The Design generally of Iron Bridges of very large Span for Railway traffic." By THOMAS CURTIS CLARKE, M. Inst. C.E.[1]

SINCE the year 1863, when the late Mr. Zerah Colburn, M. Inst. C.E., read a Paper on the subject,[2] no communication has been submitted to this Institution relative to the construction of iron railway bridges of long spans as practised in America. Although the title of this Paper does not limit its scope to American construction, yet inasmuch as the physical features of American rivers require, and have led to the development of, spans longer than are usual in other countries, it has been thought best to begin by describing American practice, and afterwards to generalise so as to cover the whole subject indicated by the title.

At the time Mr. Colburn's Paper was written, the longest iron span in America was the central tube of the Victoria Bridge at Montreal, 330 feet in the clear. Since then trussed girder bridges have been built up to 515 feet clear span, which is the length of the channel span of the Cincinnati Southern railway bridge over the Ohio, at the City of Cincinnati, although the span between the masonry is only 500 feet. This is the longest railway girder yet constructed. The Kuilenburg bridge in Holland is the next longest, being 492 feet in clear span. The side arches of the steel bridge over the Mississippi at St. Louis are 502 feet in the clear, and the central one is 520 feet in clear span between the masonry.

All these American bridges are "pin connected," this style of construction being preferred by American engineers for spans exceeding 100 feet, on account of the mathematical certainty with which the strains can be calculated; the ease, celerity, and economy

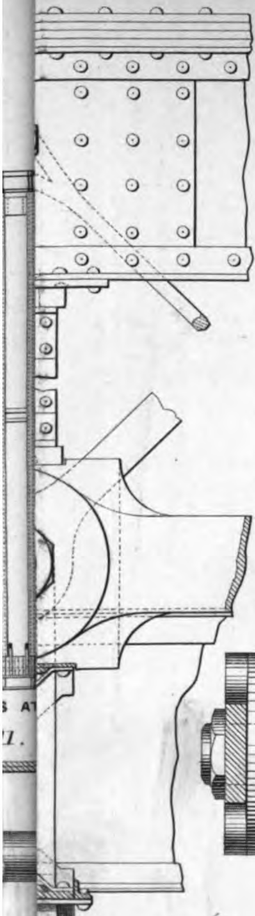
[1] The discussion upon this Paper occupied portions of two evenings.

[2] *Vide* Minutes of Proceedings Inst. C.E., vol. xxii.. p. 540.

of erection, which for rivers subject to sudden floods is a ma of vital importance; and lastly, because it is believed that parts of a bridge can be more strongly united in this way t by riveting. Add to this a considerable possible reduction dead weight of iron over that of riveted construction, and power of being able to calculate the deflection or camber beforeh with certainty, and reasons enough have been given why construction is preferred by American bridge designers. limits of this Paper do not allow of any discussion of the meri the two systems. Suffice it to say, that in America, bridges ceeding 100 feet span generally have their parts connected pins; below that length riveted lattice and plate girders gaining favour, and becoming more and more extensively use their size and weight admit of their being transported from workshops to the place of erection, either whole, or at least in more than two or three pieces, thus reducing the riveting position to a minimum. There are, it is true, a few example riveted lattice girders in America up to 180 feet span; but order to compete with the pin-connected bridges, their desig have reduced the amount of iron far below what is desira and their workmanship is much inferior to the best Eng practice.

Two of the latest and best examples of American long-s iron bridge construction have been chosen for illustration. channel span of the Cincinnati Southern railway bridge ac the Ohio river at Cincinnati (Plate 7):—515 feet between points of bearing, is selected as an example of a well desig girder of the type now most approved in America, and erec in the usual manner by temporary stagings of timber. The ot example is the bridge of three spans of 375 feet each, carry the same railway across the Kentucky river (Plates 8 and Both these bridges are noteworthy for their economical desi and for their comparatively small amount of dead weight.

According to the American custom the engineer of the Cinc nati Southern railway invited public tenders for both the desi and construction of these bridges, furnishing sections of the rive with a general specification (Appendix II.) giving the loads to provided for, and the maximum strains allowed upon the iro The length of span was left to the designer in the case of th Kentucky bridge, but was prescribed in the specifications of th Ohio bridge. The specifications also provided for a certai quality of iron, and tests were required to be made before ente



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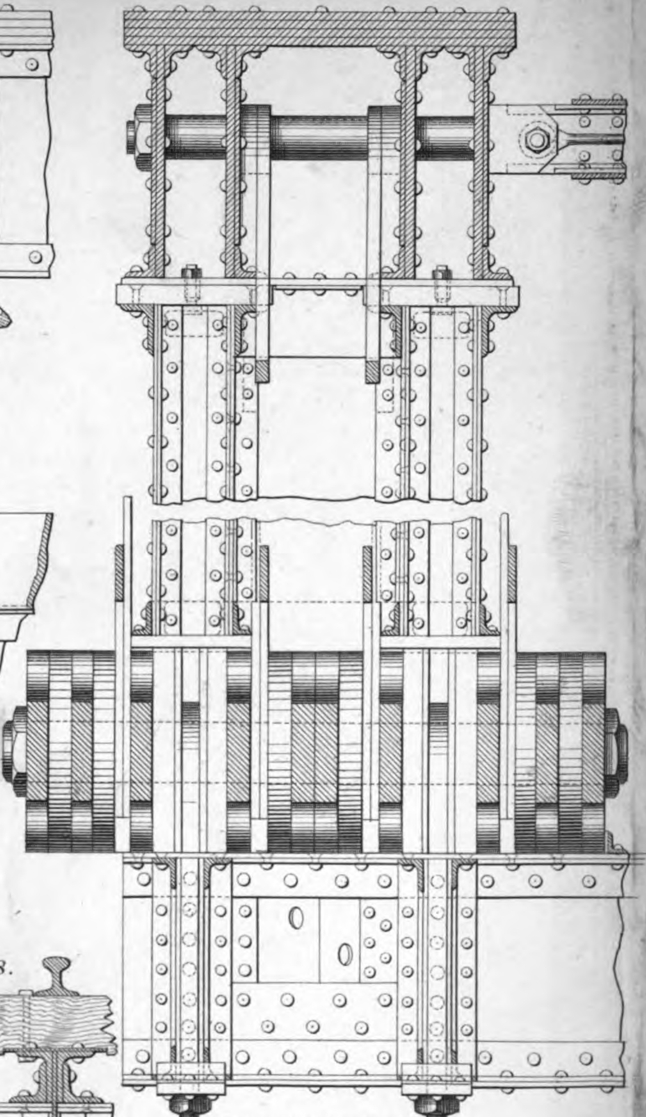
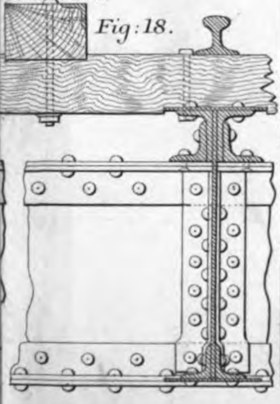


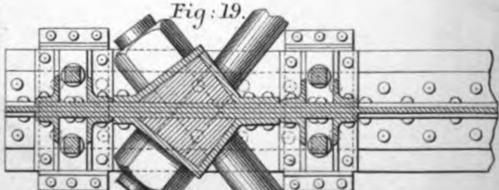
SECTION AT CENTRE POST.

Fig: 18.



ITUDINAL GIRDERS FOR RAILS.

Fig: 19.



HORIZONTAL SECTION OF CROSS GIRDER FOR RAILS.

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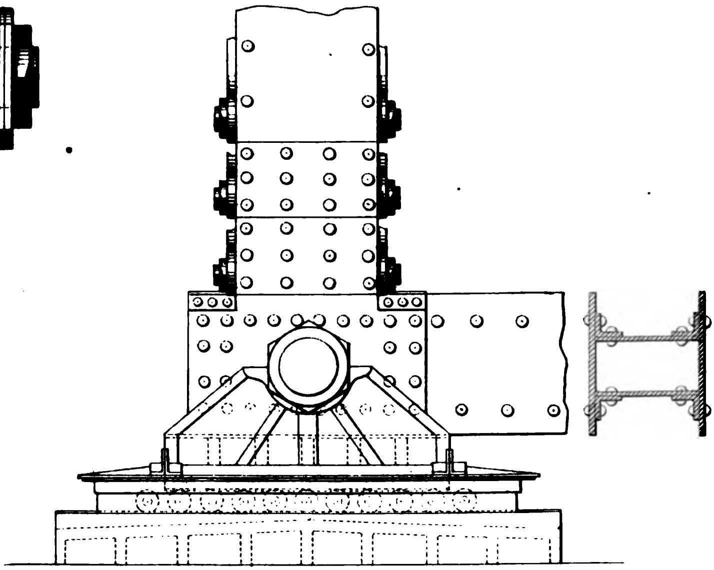
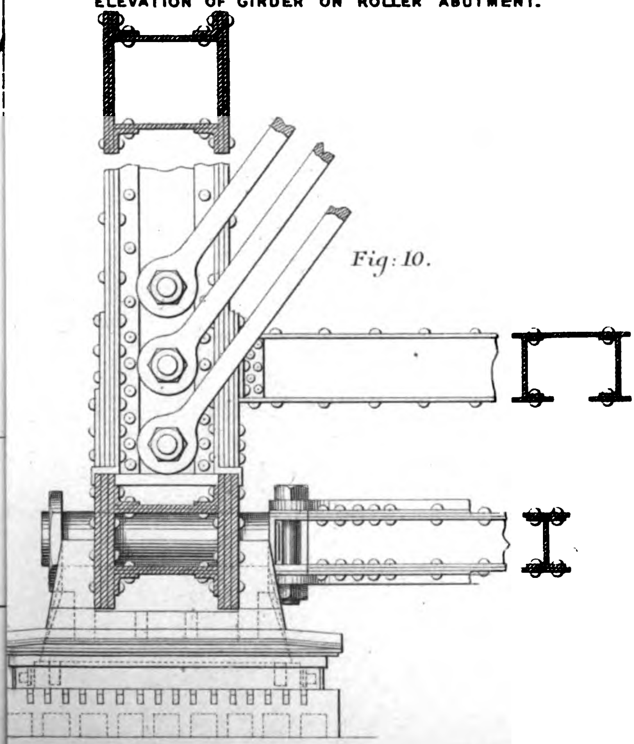
Fig: 9.



ELEVATION OF GIRDER ON ROLLER ABUTMENT.

Fig: 10.



TION OF GIRDER ON ROLLER ABUTMENT.

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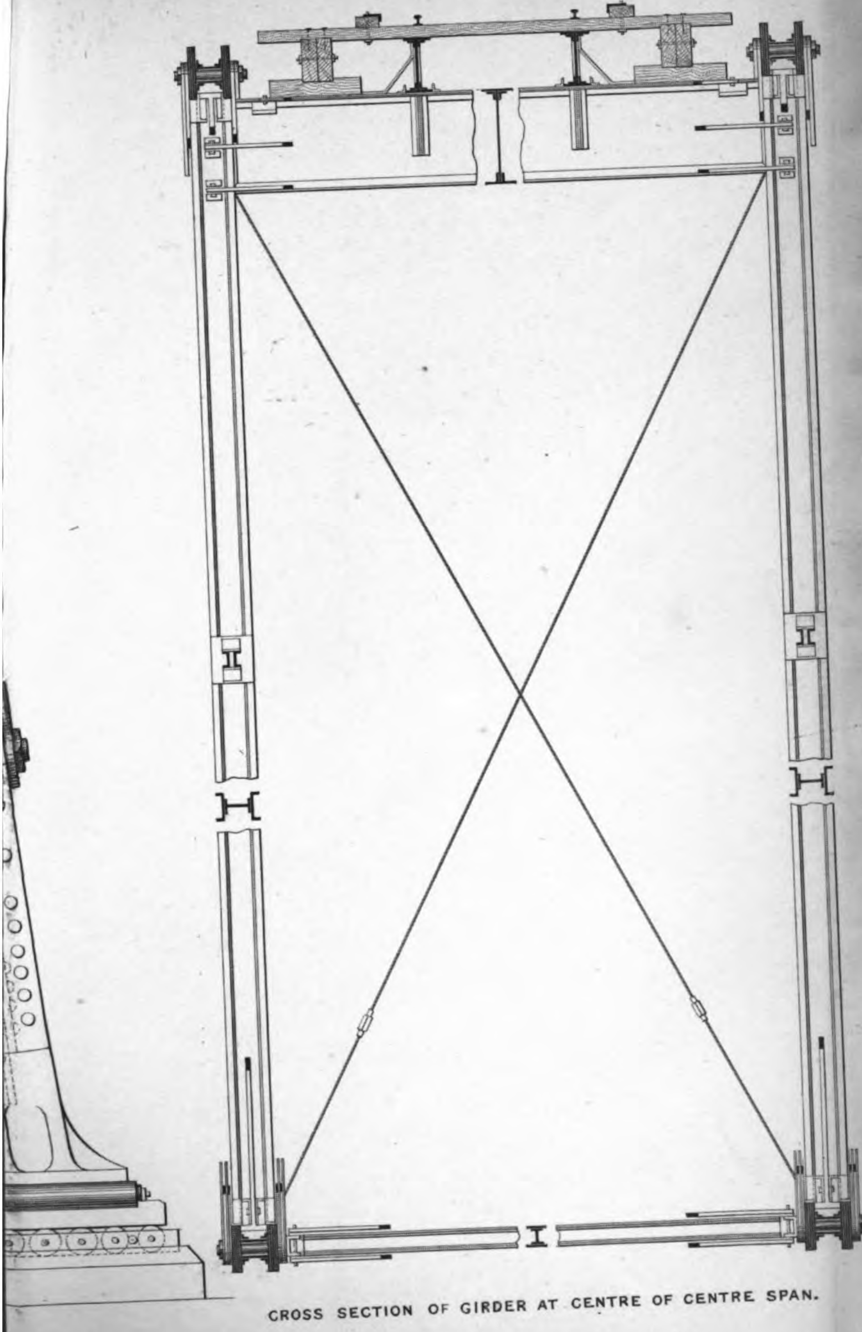
Fig: 4.



CROSS SECTION OF GIRDER AT CENTRE OF CENTRE SPAN.

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ing upon construction. These tests were extended to cover proportions of eye-bars, and the strength of various sections of column-struts, and gave some useful information not accurately known before. The results of these tests are printed in Appendix III.

The bridges of the Ohio, which is a navigable river flowing past the boundaries of several states, have to be constructed in accordance with Acts of Congress of the United States, fixing their minimum spans and heights, and all the plans have to be approved by the United States engineers. The requisitions laid down by this body demanded a channel span of 500 feet in the clear between the bases of the piers, or 515 feet between the points of support, placed at a height of 105 feet above low water in the river, which is 10 feet deep, while the river rises 62 feet in times of floods.

The lower line of staging was erected in December 1876. The surface of the river was covered with a continuous sheet of ice. A sudden thaw and rise of water of 15 feet washed away the lower staging. Operations were suspended until June 1877, when the staging was rebuilt. The bottom of the timber trestles stood on the rocky bed of the river. About 500 feet above the line of the bridge a timber crib was sunk and filled with stone. From this a fender of coal-barges chained together was let down to each pier, forming a triangle, of which the crib was the apex and the two piers stood at the lower angles. This fender protected the trestle from being struck by boats or drift-wood, it being arranged to rise and fall with the water.

The ironwork of this channel span of 515 feet had been submerged by a freshet and required washing. The iron was cleaned, elevated 100 feet by steam power, run forward over two spans of 300 feet each, and then hoisted into position by two travellers on the top staging, worked by hand. This was all done in twenty-four and a half working days by an average force of sixty men. The span was swung clear of its staging by knocking out folding wedges, and then required thirty men for six days more to adjust it, put on cross girders and track stringers, and lay the ties and guard timbers.

The span was tested by running over it seven locomotives and four loaded platform cars, their combined weight amounting to 431 tons.

The centre deflection of the east truss was $2\frac{3}{32}$ inches; the permanent set was $\frac{1}{16}$ inch.
" " " west " 2 " " " none.

The cost of this span was as follows:—

	£ s.	£
1,176 tons iron at shop . . .	30 12	36,000
Freight		1,300
Staging	3,500 0	
Erection	600 0	
		4,100
Painting		500
		£41,900

It will be observed that the cost of erection, including staging, is about £3 10*s*. per ton, while that of the Kentucky river bridge, to be described hereafter, which was erected without staging, was about £2 10*s*. per ton. Had the Ohio river span been erected without the staging and crib being carried away by the freshet, the cost of its erection would probably have been a little less than that of the other.

This bridge was designed and executed by Mr. J. H. Linville, Member of the American Society of Civil Engineers, who has built five first-class bridges, having channel spans of from 319 feet to 515 feet in length. This last one represents the best practice yet attained, and does great credit to its accomplished designer.

The Kentucky river bridge (Plate 8) consists of three spans of 375 feet each, carrying the Cincinnati Southern Railway across a limestone cañon at a height of 280 feet above the bed of the stream. The piers are of stone to a height of 60 feet, which carries them a little above the highest recorded floods, and of iron for the rest of their height. As the floods are very sudden, it was decided to adopt a plan of construction obviating "false works" or stagings of timber, which might be swept away by freshets, or knocked down by rafts, which pass by during floods at a speed of 7 or 8 miles per hour.

In 1854 the late Mr. J. A. Roebling began to build a suspension bridge, of 1,236 feet span, across this chasm, but the work was abandoned for want of funds, after two towers and two sets of anchorage had been constructed. Mr. Shaler Smith, the engineer whose design was adopted, took advantage of the existence of these towers by bolting the first panel of his bridge, on each side, to them, and then corbelling out panel by panel. The towers were calculated to be strong enough to carry 196 feet of projecting spans. At this point temporary towers of wood were built, which gave an intermediate support. The corbelling out process was continued until the shore spans each reached the main iron piers, which were

built up simultaneously, so that the two met in mid-air. These piers had been built on rollers, as it was impossible to foresee the temperature at which the junction would take place. Each pier, weighing 200 tons, was moved horizontally until a junction was made. Each half of the centre span was then corbelled out as before, until they met in the centre. At this stage of the work, the upper chords being in tension, and the lower in compression, the former were nearer to each other than the latter, the gaps being as follows:—

Upper chord, east gap	3 inches.
" " west "	2 "
Lower " east "	4 "
" " west "	5 "

The gap of 2 inches in the west upper chord was first closed by screw-jacks, which had been placed between the ends of the lower chords and the abutments on the shore spans, and by moving the piers together. This left a gap of $1\frac{1}{8}$ inch between the ends of the east top chord. At mid-day the temperature of the air being 70° Fahr., all the horizontal lateral rods tending to draw these ends together were screwed up and the counters slackened. The contraction of the lateral rods drew the gap together. On the following morning, at a temperature of 40° Fahr., the gap had closed, and the top chord connections were riveted up. The contraction due to temperature had by that time withdrawn the shore ends of the lower chords from the screw-jacks $\frac{3}{4}$ inch. These were then screwed home so as to take up this space, and by mid-day the lower chords had expanded until the gap in the east chord was closed, which was then riveted up, and by a similar process the gap in the west chord was closed.

It will be observed that up to this time this bridge was a girder of 1,125 feet long, continuous over three spans. But it had been foreseen that a continuous girder would not answer for this situation, because while the abutments on the cliffs were stationary, the iron piers would constantly rise and fall with changes of temperature, and so vary the strains on the web system. It was therefore determined to hinge the shore spans at points 75 feet from the piers, leaving a centre girder 525 feet long, supported by piers 375 feet apart. By this means the shore spans were practically reduced to 300 feet each, one end resting on the abutment, and the other on the overhanging end or cantilever of the centre span.

The final operation, therefore, was to cut the lower chords of the shore spans at points previously determined by calculation, at which points tenon joints had been made and temporary rivets

inserted. These rivets were cut out, and the mean motion of the several joints was only $\frac{5}{16}$ inch, and the change in the levels barely perceptible. This proves the accuracy of the method used to determine the point of contrary flexure, which was to work out the strains panel by panel, as in calculating discontinuous spans.

To avoid ambiguity in the web strains at the hinging points, both of the web systems of diagonal rods were consolidated into one member at the point of contrary flexure, and separated again after the hinge was passed. When the bridge came to be tested, it was found that the movement of the lower chord tenons under the passing load was $1\frac{1}{2}$ inch. This shows how great may be the strains concealed in the web system of a continuous girder of large span at the point of contrary flexure. The whole theory of continuous girders depends upon the assumption that the modulus of elasticity of every part of the ironwork is uniform. In the construction of this bridge the greatest efforts were made to secure this. Iron mixtures were prescribed in the puddling furnace, and in the rolling mill pile. Every plate was tested, and all bars paired together in construction by their modulus, and the workmanship was exceptionally exact. But in spite of all this care, the moduli varied from 20,000,000 lbs. to 28,200,000 lbs.[1] During erection the ends of the trusses began to vary in height, and the variation in length between the east and west chords was 1 inch in 1,125 feet. This shows clearly the practical impossibility of getting uniform moduli of elasticity.

The results of the official test were as follow:—

1. End spans loaded with 277 tons, centre span unloaded; centre deflection 1·52 inch. Upward movement of centre span 2·83 inches.

2. Centre span loaded with 331 tons, end spans unloaded; centre deflection 3·5 inches. Upward movement of cantilever 1·58 inch.

3. All spans loaded 814 tons in 904 feet; centre deflection of centre span 1·62 inch.

4. Horizontal movement of top of pier when a heavy train, 904 feet long, moving at 26 miles per hour, was stopped in 104 feet = $\frac{1}{2}$ inch.

This bridge expands from the centre, each way, bending the tops of the piers toward the shores; the greatest observed movement

[1] This variation of 40 per cent. is far too great for bars of the same section, and results from the iron having been made at different works, and from different qualities of pig. Bars of similar section, of double refined iron made at Phœnixville from the same ore mixtures, do not vary more than 5 per cent. in their moduli of elasticity.—T. C. C.

has been ½ inch each way. There are rollers on the abutments and the greatest observed motion has been 3 inches.

The erection began on the 16th of October, 1876, and ended on the 20th of February, 1877, thus occupying four months and four days. At no time did the staff exceed sixty men, and the average number was about fifty-three. The cost of the erection of this bridge was very small, considering the difficulties of handling and distributing the iron. The total cost of erection, including plant, was as follows:—

	£	£
Cost of making roads, etc.	2,000	
Hauling and distributing iron	500	
Erection proper	4,000	
		6,500
Cost of 1,631 tons of iron at £21 7*s*.	34,833	
Freight on ditto	3,722	
		45,055
Cost of foundations:		
15,000 cubic yards at 7*s*. 5*d*.	5,573	
Cost of masonry:		
12,635 cubic yards at £2 8*s*.	30,218	
		35,791
		£80,846

The weight of iron was thus distributed:—

	Lbs.	Lbs.
In 300 feet	716,554	
„ 75 „ cantilever	230,000	
In 375 feet south span		946,554
„ 375 „ north „		946,554
„ 375 „ centre „		962,163
„ two piers		799,000
		3,654,271

The dimensions are as follows:—

Total length of ironwork	1,138 feet.	
„ „ of iron between pier centres	1,125 „	
Depth of truss	37½ „	
Width „	18 „	
Iron pier at base	28 feet by 71½ „	
„ „ top	1 foot „ 18 „	
Height of pier	177½ „	
Masonry pier	42 feet by 120 feet by 71 „	high.
Height of rails above low water	275½ „	
„ „ „ river bed	279½ „	
Flood rise	57 „	

Seven floods took place during the construction of the bridge, their rise varying from 26 to 47 feet.

This is not only one of the boldest and most original pieces of bridge engineering in America, but when judged by the crucial test of accomplishing a great deal at the least possible cost, it stands very high among engineering structures all over the world,

and its design and execution reflect the highest credit upon its engineer, Mr. C. Shaler Smith, Member of the American Society of Civil Engineers.

It has been said that both these bridges were conspicuous for economy of design. Economy of design means the art of getting rid of dead weight. It is of comparatively little importance in short spans; but it is vitally essential in very long ones, as the proportion of dead weight to live load increases in a much higher ratio than the lengths of the spans, even in the best designs. As the limit of span is that distance at which the bridge would break from its own weight alone, the less dead load there is to carry, the farther may that limit be placed.

Economy of design is attained by two entirely distinct processes. First by proportioning all the parts of a bridge with a similar factor of safety, and then combining those parts into a whole so that the structure itself will be as strong as the parts of which it is composed; and secondly, by using such proportions of height of girder, length of panel, and combination of parts; also such width apart between the girders, and such methods of bracing the two into a structure able to resist wind pressure or shocks from derailed trains, as will accomplish the first requisite with the least quantity of metal.

The problem is too difficult to be solved *à priori*, and can only be done by a tentative process. To show how it has been accomplished a table has been prepared (see pp. 194 and 195), giving the weights of iron and other important data of the most conspicuous examples of long span railway bridges constructed in Europe and in America.

In order to see how much dead weight can be saved by judicious design, compare No. 2, Mr. Linville's bridge built in 1864, of 319 feet span, weighing 484 tons, with No. 4 built in 1870, a bridge of 342 feet span, weighing 338 tons. The older bridge has, it is true, upper chords and posts of cast iron, and hence is much more massive than if the same members were executed in rolled iron; but a considerable difference of weight also comes from difference of proportion. The older bridge has panels 12 feet 3 inches long, and its height is but $\frac{1}{12}$ of its span. The newer bridge has panels 15 feet $6\frac{1}{2}$ inches long, and its height is $\frac{1}{10}$ of its span.

Comparing Nos. 6 and 7, No. 6 of 368 feet span weighs 497 tons; No. 7 of 375 feet span weighs 425 tons. Here the difference is not due to the height, for the heavier bridge is $\frac{1}{8}$, while the lighter one is $\frac{1}{10}$ of the span. It arises from the unnecessary width of No. 6, which requires a heavier lateral system of bracing, and to

four trusses being used where two would have answered, thus making double the number of joints and connections; and dates from the works of Whipple in 1847, of Bollmann in 1852, and of Albert Fink in 1854, while if the Saltash bridge, No. 13, be compared with the Britannia bridge, No. 14, it will be noticed that the greater economy of the former is due to the metal being concentrated along the lines of strain, which is claimed as peculiarly American practice, but it was only introduced by Brunel in 1859.

The Mayence bridge, No. 5, on the Gerber system, is similar to the Saltash bridge, and compares very favourably in weight with the newest American example, No. 4. But the most remarkable difference in the amount of metal used to attain nearly similar results will be seen by comparing the Kuilenburg bridge, No. 15, of 492 feet span, with the Ohio bridge, No. 16, of 515 feet span, which has been described. The dead load of the Kuilenburg bridge, together with a live load of 3,000 lbs. per lineal foot, strains the iron 14,560 lbs. per square inch. Although this bridge carries but one line of rails, it is wide enough for two; to increase the width of the Ohio river bridge to 30 feet would add about 450 tons of iron to its weight. To carry this additional weight and that of a load of 3,000 lbs. per lineal foot would increase the strains on its chord members from 10,000 lbs. per square inch to 13,900 lbs. per square inch; so that it would even then be stronger than the Kuilenburg bridge. The Kuilenburg bridge contains 2,234 tons of metal. The Ohio bridge, 30 feet wide, would have 1,626 tons; 608 tons less than that of a bridge 23 feet less span.

The following table shows the weight of each in detail:—

KUILENBURG and OHIO SPANS COMPARED for WEIGHT.

Where used.	Kuilenburg. 492 Feet Span. Tons of 2,240 lbs.	Ohio River. 515 Feet Span. Tons of 2,240 lbs.
Top and bottom members	1,203	594
Diagonal ties	243	162
Posts	340	165
Lateral and wind bracing	268	65
Transverse floor beams	54	49
Longitudinal „ „	44	111
Bed plates, rollers, &c.	82	30
Total metal	2,234	1,176
Add 8,000 cubic feet pine } „ 9,500 „ „ fir }	306	Ties, guards, and rails 60
„ plates between rails	350	..
Total dead load	2,890	1,236

It strikes an engineer accustomed to American practice with surprise that the designer of the Kuilenburg bridge, apparently not satisfied with the excessive dead weight of the iron work, has piled upon the structure 656 tons of timber and iron plates. The engineer of the Ohio river bridge was able to confine the weight of the track and guard timbers to 60 tons.

The difference in the weight of the upper and lower members is remarkable, and is owing in a great measure to the arched form of the upper member of the Kuilenburg, which increases the strains toward the ends. The short panels, 13 feet 1 inch, of the Kuilenburg require nearly double as many sets of connections as the 29 feet 9 inch panels of the Ohio bridge; the connections by riveting also necessitate much more dead weight than connections by pins and eyes.

Detailed calculations of strains and weights made by the Author have convinced him that the economical limit of a single track iron girder span lies not far beyond 600 feet. The actual limit, beyond which it could be made and have strength enough left after carrying itself, to support two locomotives and a train of loaded coal wagons, lies considerably beyond this distance.

The difficulty in designing girders of very long span is to get width enough to resist wind pressure. Such girders require to be very high. The Ohio bridge would be of better proportions if its height had been $\frac{1}{8}$ of its span, or 64 feet. To give it as much lateral stability as it now has, the width in that case should have been 26 feet. This could have been done probably without much increase over its present weight. But add 100 feet to it and preserve these proportions, and the span would be 615 feet; height 77 feet; width 31 feet. This is about the limit for two main girders.

Suppose the same proportions for a double track bridge, with four girders of 715 feet span, height 90 feet, width 36 feet, or better 40 feet (Fig. 3), and the cross sections and side view will be thus (Fig. 1 and 2). Here is a structure possessing lateral stability as well as strength to resist the forces of gravity. The weights of such a bridge for two tracks have been carefully worked out by the Author. They amount to 4,900 tons of metal. The difficulties and risks of erection are very great for such a structure as this; hence it is said that although practicable it is not as economical as other forms. These other forms are :—Fig. 4. Cantilevers with connecting span. Fig. 5. Stiffened suspension bridges. Fig. 6. Arches.

The suspension bridge, stiffened either by bracing the chains or ...nded girder, is undoubtedly the most economical struc-

FIG. 2.

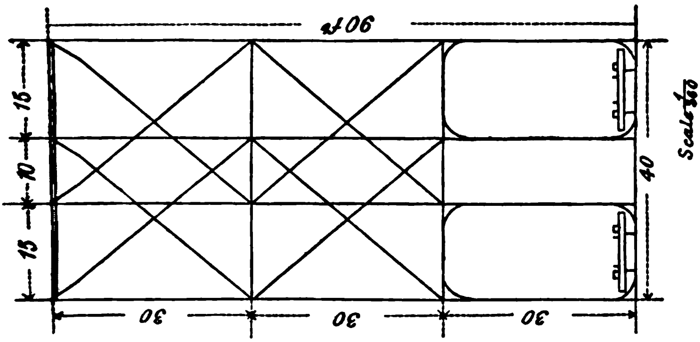
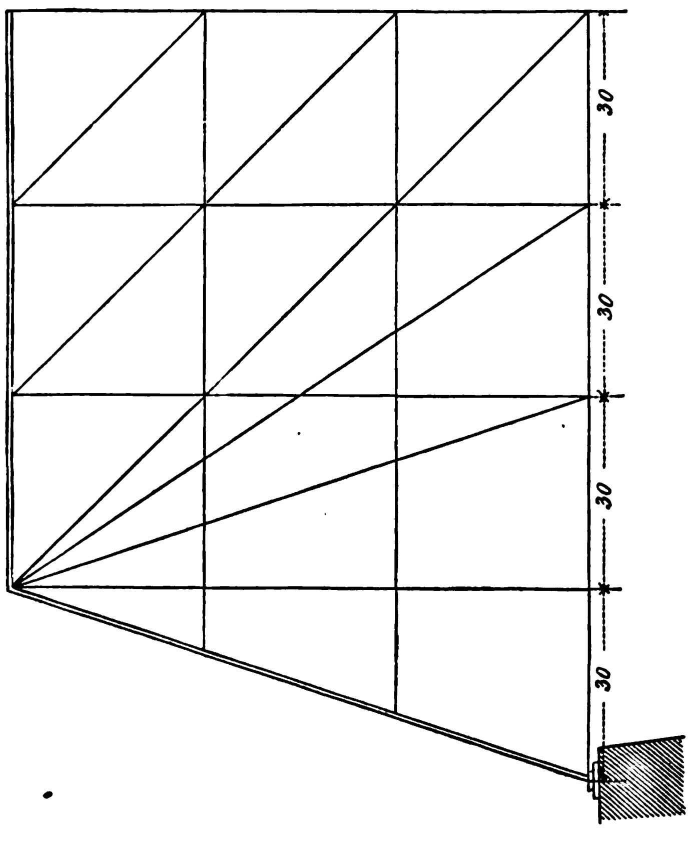


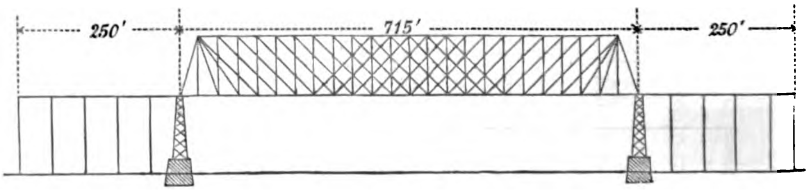
FIG. 1.



ture possible for long spans between towers. Whether it will be the most economical *per se*, depends upon whether the banks are high or low. If the banks of the river are as high as the level of the bridge, as at Niagara, it will be the most economical form possible for spans of 700 feet and upward.

The "Point" bridge at Pittsburg, U.S., of 795 feet span, built in 1877 (E. Hemberle, Member of the American Society of Civil Engineers, being the engineer), although a carriage bridge, is strong enough to carry a railway train of 650 tons weight, with a factor

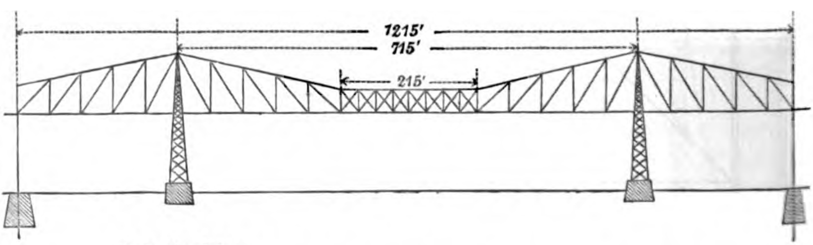
FIG. 3.



Data of weights—

715 feet main girder and piers	4,900	tons.
500 feet viaduct	500	„
	5,400	„

FIG. 4.



Data of weights—

1,215 feet cantilever, including piers and anchorages . . . . . . 4,340 tons.

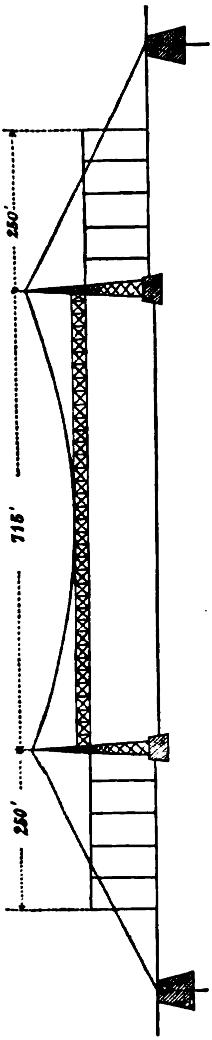
of safety of 5. It weighs a little less than 1 ton per lineal foot between the towers.

The Niagara bridge of 821 feet span, built by John A. Roebling in 1855, has its wooden suspended truss badly decayed, and it is proposed to replace it in iron during the present year. Plans have been submitted by a commission of engineers, and if these are carried out the weight of iron between the towers will be a little more than 800 tons. It will then be able to support a railway train of 500 tons with a factor of safety of 4.

Where the banks are low, the increased length of the back chains will add to the cost, and other forms may be preferable.

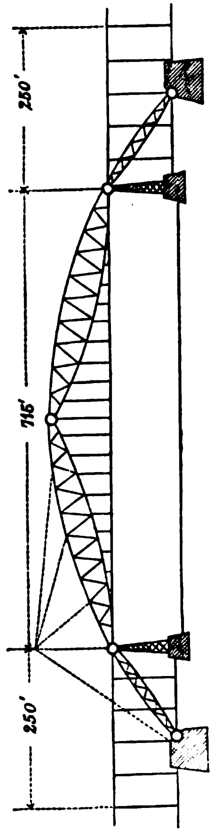
The cantilever, with connecting span, will never, under any circumstances, be as economical of metal as the stiffened suspension bridge. It shares with it the defect of having its stability

FIG. 5.



Data of weights—
715 feet suspension girder and back chains . . 3,000 tons.
500 feet viaduct . . . . 500 „
3,500

FIG. 6.



Data of weights—
715 feet of arch and shore abutments . . . . 2,800 tons.
500 feet viaduct . . . . 500 „
3,300 „

Scale $\frac{1}{5000}$.

dependent upon anchorage built in masonry, and practically inaccessible. Its continuity being cut in the centre, it has to be very wide in order to resist wind pressures, which in long spans become

an important factor in modifying the shape of the structure. No large cantilevers have yet been built, and their success on an extensive scale is problematical.

The arch bridge remains, and that form consisting of two lunettes hinged at the centre, invented by James B. Eads, M. Inst. C.E., appears to possess many advantages. Where the headway required allows of its being placed below the track of bridge, its economy becomes manifest. Where it is all above the line of rails, and where the banks are low and the level of rails high, its economy is greater than that of almost any other form, as it reaches the points of support sooner than any other form can do.

To illustrate this point, estimates of weights have been made for a double line of rails with a moving load of 3,400 lbs. per foot, wind pressure 40 lbs. per square foot, iron to be strained 10,000 lbs. per square inch. Length of spans 715 feet; height above the level of the shore, 100 feet. The weights include two iron piers, and 500 feet of viaduct have been added to Nos. 1, 3, and 4, to equalise them in length with No. 2.

The erection of the suspended girder would not be difficult, and no staging would be required. The same may be said of the cantilever, and of the Eads arch, which would, for the purposes of erection, be temporarily made into a cantilever by erecting a timber tower, and suspending it by iron tie rods, as shown by the dotted lines. The erection of the quadrangular girder would always be expensive, and probably involve much risk. In some places it would be impossible.

Nothing has been said in reference to economizing weight by the use of steel. At present its cost is too high, and its strength is too variable. When more is known about steel, it may be used in very long spans. All the compressive members in designs 2, 3, and 4 might be made of steel with economical results, and its use in them would require no new processes of manufacture. Steel eye bars have not yet been made, at least to the knowledge of the Author.

The workmanship of long span bridges in the United States is generally first class. All abutting surfaces are planed, bored, or turned. The limit of error in links is $\frac{1}{64}$ inch, which is attained by drilling both holes at the same time on a machine whose bed is of wrought iron, hence expanding equally with the bar. Eye bars are proportioned by rules derived from many experiments. A Paper read by C. Shaler Smith, Member of the American Society of Civil Engineers, gives the latest and best practice. Holes are punched, and riveting is done by steam or hydraulic power, and

three hundred rivets per hour is not an unusual performance. In riveting at the site, the holes are punched small, and reamed and drilled to the proper fit.

The price of bridge work in the United States has fallen every year with the lower price of iron. The following table gives an approximation to the prevailing rates for American pin-connected bridge work :—

Years.	Prices per Ton.			Remarks.
	American Pig.	American Bar Iron.	American Bridge Work.	
	£ s.	£ s.	£ s.	
1870	6 13	15 16	40 6	Prices of riveted lattice plate girder work £2 10s. to £3 less than pin-connected.
1871	7 1	15 14	38 2	
1872	9 16	19 10	37 7	
1873	8 11	17 6	34 7	
1874	6 1	13 12	30 10	
1875	5 2	12 4	27 4	
1876	4 9	10 8	24 11	
1877	4 0	9 10	20 16	

The Paper is illustrated by a series of diagrams and numerous sketches from which Plates 7, 8 and 9 and the woodcuts Figs. 1 to 6 have been engraved.

[APPENDIX

APPENDIX I.—TABLE of TUBULAR and GIR
Constructed of iron.

No.	Date of Erection.	Where Built.	Name of Engineer.	Clear Span in Feet between Points of Bearing.	Tons of Iron (2,240 lbs.).	Dimensions in Feet: Width between Centres of Girders.		Panels: Number.	Panels: Length.		Panels: Heig	
						Ft.	Ins.		Ft.	Ins.	Ft.	Ins.
1	1877	Susquehanna river, Havre de Grâce, U.S.	Phœnix Bridge Company .	307	215	16	0	17	18	2	35	0
2	1864	Ohio river, Steubenville, U.S.. . .	J. H. Linville .	319	484	16	6	26	12	3	28	0
3	1859	St. Lawrence river, Montreal, Canada .	Robert Stephenson . . .	330	686	16	0	Ends Centre			22 30	0 0
4	1870	Ohio river, Parkersberg and Belleaire, U.S.	J. H. Linville .	342	338	18	0	22	15	6	33	0
5	1862	Rhine river, Mayence	Gerber . . .	345	359	15	1	13	25	4	24 49	6 2
6	1870	Ohio river, Louisville, U.S.. . .	Albert Fink .	368	497	30	0	24	15	4	46	0
7	1877	Kentucky river, Dixville, U.S.. . .	C. Shaler Smith	375	425	18	0	20	18	9	37	6
8	1870	Ohio river, Louisville, U.S.. . .	Albert Fink .	396	623	30	0	28	14	2	46	0
9	1856	Vistula river, Dirschau	Lentze . . .	397	838	21	8	Close Lattice			23	6
10	1848	Conway	Robert Stephenson . . .	400	1,112	15	0	Tube			25	6
11	1871	Ohio river, Cincinnati, U.S. . . .	J. H. Linville .	415	830	19	0	20	20	9	41	6
12	1861	Inn river, Passau .	..	420	327	15	0	23	14	0	27	0
13	1859	Saltash	I. K. Brunel .	455	945	17	0	12	38	0	30 60	0 0
14	1850	Menai Straits, Britannia, Wales. .	Robert Stephenson . . .	460	1,553	15	0	Tube			30	0
15	1868	Lek river, Kuilenburg, Holland .	G. van Diesen .	492	2,234	30	4[1]	38	13	1	26 65	3 7
16	1877	Ohio river, Cincinnati, U.S. . . .	J. H. Linville .	515	1,176	20	0	20	25	9	51	5

[1] ... noticed that the width of the Kuilenburg bridge, 30 feet 4 inches, is sufficient for two lines of w... ...e line has been laid.—T.C.C.

IDGES for SINGLE TRACK RAILWAY.
Spans exceeding 300 feet.

s in lbs. per eal Foot.		Strains, in lbs. per Square Inch of Area.								
		Tensile.		Compressive.						
d of nd er k.	Live Load of Engines and Cars.	From Constant Load only.	From Total Constant and Moving Loads.	From Constant Load only.	From Total Constant and Moving Loads.	Test Load.	Centre Deflection.	Dead Load of Iron, Timber, &c.	Deflection of span from its own weight on removal of scaffolding.	Remarks.
						Tons.	Inches.	Tons.	Inches.	
50	2,240	4,900	10,000	4,400	9,000	..	..	..	..	All rolled iron except joint blocks. Pin connections; quadrangular girder.
00	3,000	5,700	10,000	3,100	6,000	..	..	..	..	Top chords and posts of cast iron, rest of rolled iron. Quadrangular girder.
00	2,240	7,680	11,200	6,060	8,900	385	$1\frac{3}{8}$	743	$7\frac{1}{2}$	Tubular girder.
00	3,000	4,545	10,000	3,636	8,000	..	..	..	..	All rolled iron. Pin connections; quadrangular girder.
90	2,890	7,120	11,600	..	..	450	$\frac{27}{100}$	719	$2\frac{1}{4}$	Pauli system; all riveted work; lenticular girder.
68	2,600	7,000	Chords 12,000 Diag. 10,000	3,500	6,000	200	1	..	..	Top chords cast, rest rolled iron. Pin connections; quadrangular girder.
700	2,037	5,080	10,000	4,570	9,000	331	$1\frac{63}{100}$	..	..	All rolled iron. Pin connections; quadrangular girder.
168	2,600	7,400	Chords 12,000 Diag. 10,000	3,700	6,000	200	$1\frac{1}{8}$	..	..	Top chords cast, rest rolled iron. Pin connections; quadrangular girder.
160	2,128	7,220	9,720	7,220	9,720	..	..	..	..	All rolled iron, one line of rails only taken. Close lattice with posts two spans continuous.
450	2,240	9,200	12,400	..	..	$300\frac{1}{2}$	$2\frac{13}{100}$	1,112	$8\frac{3}{50}$	Tubular girder.
500	4,500	5,500	10,000	4,950	9,000	..	..	..	..	Carriage way on each side of railway. Pin connections; quadrangular girder.
150	2,210	6,700	13,700	5,900	12,000	165	$\frac{13}{100}$	396	$\frac{9}{10}$	Riveted lattice. All rolled iron.
500	2,240	6,650	8,960	..	..	384	$1\frac{2}{10}$	1,100	$2\frac{1}{2}$	Lenticular girder.
780	2,240	10,375	13,336	..	..	248	$\frac{68}{100}$	1,553	$11\frac{74}{100}$	Tubular girder.
200	3,000	11,800	14,560	8,200	10,000	595	$1\frac{32}{100}$	2,890	$3\frac{1}{25}$	All rolled iron, riveted. Lattice arched top.
400	1,818	7,470	10,000	6,600	9,000	431	2	1,236	See Remarks.	All rolled iron. Pin connections; quadrangular girder. Estimated camber, $4\frac{1}{2}$ ins.; actual, $3\frac{3}{4}$ ins.

APPENDIX II.—CINCINNATI SOUTHERN RAILWAY.

I. OHIO RIVER BRIDGE SPECIFICATIONS.

Superstructure.

"The superstructure of this bridge will be of iron, with the exception of cross-ties and guard timbers.

"Bidders will furnish a general plan with detail drawings and diagrams of strains for each span, otherwise their tender will not be received.

"The bridge must be not less than fourteen (14) feet in width in the clear, and not less than eighteen feet six inches (18½) in height in the clear, measuring from the top of the rail. The roadway on each span will be constructed so as to leave on the rail only one-half of the camber of the truss.

"The channel span of bridge shall not be less in width than twenty (20) feet from centre to centre of trusses.

"The draw must be geared for both hand and steam power, so as to turn either way. All machinery for the same must be furnished and put in place by the contractor.

"The hand gearing must be so arranged that two men can easily turn the draw in a calm day in two and one-half minutes.

"Steam power must be applied of sufficient capacity to turn the draw in one minute, with a pressure of 40 lbs. to the square inch.

"The bearing of the ends of the draw on the piers must be firm, so as to insure a smooth track for the passage of trains. All locks must be worked by gearing from the centre of the draw, and be self-adjusting when the draw is 6 inches out of line.

"Not more than three (3) feet and three (3) inches shall be occupied from the base of the rail to the lowest point in the superstructure of the bridge.

"*Rolling Load.*—All parts of the bridge must be proportioned to sustain the passage of the following rolling load, at a speed of not less than thirty miles per hour, viz.: two locomotives coupled, each weighing 36 tons on drivers, in a space of 12 feet; total weight of each engine and tender loaded, 66 tons in a space of 50 feet, and followed by loaded cars weighing 20 tons each in 22 feet. Weight of locomotive and tender to be distributed according to the following diagram. An addition of from 10 to 30 per cent. will be made to the strains produced by the rolling load (considered as static) in the calculation of floor beams, stringers, suspension-links, counter-rods, and all other parts of the bridge which are liable to be thrown suddenly under strain by the passage of a rapidly moving load. Vertical lateral rods and struts must be of sufficient strength to resist, in addition to the live and dead load, a pressure of wind equal to 50 lbs. per square foot.

"*Dimensions of Parts.*—The ironwork shall be so proportioned that the weight of the structure including the floor, with 125 lbs. per lineal yard added for rails, spikes, and joints, together with the above specified rolling load, shall in no part cause a tensile strain of more than 10,000 lbs. per square inch of sectional area, nor a shearing strain of more than 7,500 lbs. to the square inch. The strain in compression will be reduced with the ratio of diameter to length of post, according to the Gordon formula, with a factor of safety of one-sixth. Columns for testing shall be furnished by the contractor. Experiments will be made under the direction of the Engineer, to determine the limit of elasticity, and also the

ultimate strength of the metal to be applied in the formula. In all members subject to transverse strains the maximum compression must not be more than 8,000 lbs. per square inch.

"Shearing strain on pins must not be more than 7,500 lbs. per square inch. The strain on semi-intrados of eyes not more than 10,000 lbs. per square inch, or the compressive area not less than the section of the bar. The eye must not be less in strength than the body of the bar.

"*Quality of Iron.*—Iron used under tensile strain shall be tough, ductile, of uniform quality, and capable of sustaining 60,000 lbs. per square inch of sectional area without fracture, and 25,000 lbs. per square inch of area, with a smart blow from a hammer while under strain, without taking a permanent set. The reduction of area at breaking point shall average 25 per cent.; elongation 15 per cent.; when cold it must bend without sign of fracture from 90° to 180°.

"Iron used under compressive strain must be tough, highly fibrous, of uniform quality, and capable of sustaining 25,000 lbs. per square inch of area without taking a permanent set.

"The Engineer will have the privilege at any time to select any of the bars manufactured for the bridge, cut from the same specimen bars, $1\frac{1}{4}$ inch in diameter and 12 inches long, and submit them to the foregoing tests. Should the bars thus tested fail to stand the tests, this will be considered sufficient evidence that the iron used does not comply with the requirements of the specifications.

"All bars subject to tensile strain shall be tested by the contractor, under the direction of the Engineer, to 20,000 lbs. per square inch of sectional area, with a smart blow from a hammer while under strain, without permanent set. While under the test strains, should any bar extend or contract more or less than it should do according to co-efficients of extension and contraction previously determined from experimental tests of sample bars of the grade of iron to be used, all such bars shall be rejected, for this or any other imperfection. A variation of $\frac{1}{1000}$ of an inch per foot each way for a strain of 20,000 lbs. per square inch of area will be allowed. Bars subject to shearing strain shall be of the best quality of iron, and subject to such tests as the Engineer may desire.

"Every bid must be accompanied by six specimen bars, $1\frac{1}{4}$ inch in diameter, and 12 inches long, stamped with the name of the bidder, and to be of the quality of iron that is intended to be used. All iron used in the bridge must be equal in strength and all other qualities to the specimen bars.

"*Castings* must be made of good tough cast iron; metal not less than $\frac{3}{4}$ inch in thickness, and be subjected to the following test: A bar of iron 5 feet long, 1 inch square, 4 feet 6 inches between supports, shall bear a weight of 550 lbs. suspended at the centre. The Engineer may at any time require such specimen bars to be cast from the same metal as that used in the structure. No castings will be permitted in the bridge except in the minor details.

"*Workmanship.*—All workmanship must be first-class. All abutting joints must be planed or turned. One-sixty-fourth of an inch will be the maximum error allowed in eye bars, and not more than one-hundredth of the diameter of the pin or hole.

"In riveted work all joints shall be square and truly dressed. Rivet holes shall be spaced accurately and directly opposite each other. Rivets must be of the best quality of iron, and must completely fill the holes. The area of rivets shall not be less than the sectional area of the joined pieces.

"Ends of bars having threads upon them must be enlarged beyond the diameter of the bar, enough to make the bar full size at the bottom of the thread.

"Washers and nuts must have a uniform bearing.

"*All Iron* must be painted on all surfaces before leaving the manufactory with two coats of metallic paint and oil, and with a third coat of lead and oil after the structure is erected.

"TIMBER.

"The timber shall be of white or yellow pine, or other kind of timber approved by the Engineer, free from wind shakes, large knots, decayed wood, sap, or any defect that will impair its strength or durability. No sap angle will be allowed. All timber must be inspected by the Engineer and none used without his approval. All framing must be done in a thorough and workmanlike manner.

"*Ties* must be of the best quality of white oak, not less than 13 feet in length. Must be placed one foot from centre to centre, and every alternate tie must be fastened to the stringer at both ends:

"*Guard Timbers*, 6 inches by 8 inches, must be placed about 1 foot outside the rails, must be closely notched 1 inch on each tie, and bolted to the ties every 4 feet.

"*Angle Irons*, three (3) inches by three (3) inches by three-eighths ($\frac{3}{8}$) of an inch, must be spiked to the guard timber every 18 inches alternately on the top and side. The spike holes must be counter-sunk to receive the heads of the spikes.

"RIVER NAVIGATION.

"The river must at all times during the construction and erection of the bridge be kept free for navigation of steamers, coal-fleets, tows of all kinds, and rafts.

"All coffer-dams and other obstructions must be removed by the contractor, leaving the river entirely unobstructed, except the actual space occupied by the masonry.

"RISKS.

"The contractor shall take all risks from floods and casualties of every description, and must furnish all materials and labour incidental to or in any way connected with the construction and erection of the bridge.

"TIME.

"The bridge hereby contracted for shall be commenced on or before and be completed according to the foregoing specifications in compliance with the attached contract by .

"The word 'Engineer' shall mean the Consulting or Principal Engineer, unless otherwise expressed.

"TESTS.

"Before the final estimate is paid, a thorough test of the bridge will be made by the Engineer, by loading each span with such rolling load, at such rate of speed, as described under the head of rolling load, and also by causing the load to remain on each span for the space of one hour or more. The deflection of each span during the tests made by the Engineer must not be more than the following:

"For the five hundred and twenty feet span five (5) inches.

"For each of the three hundred feet spans three (3) inches.

"For the one hundred and ten feet span one and one-tenth ($1\frac{1}{10}$) inch.

"For each span of the three hundred and seventy feet draw one and eight-tenths ($1\frac{8}{10}$) inch.

And each span must return to its original camber when the load is removed.

"MONTHLY ESTIMATES.

'neer will establish from the amount of the bid a scale of prices is for the estimation of the value of the work done each month.

"CHANGES.

"The Trustees reserve the right to change the depth of the foundations, and sizes of the piers, and abutments, as described in the foregoing specifications."

APPENDIX III.—EXTRACT FROM REPORT OF CONSULTING ENGINEERS.

TESTS OF IRON COLUMNS AND STRUTS.

"According to the articles of specification for bridges and trestles relating to dimensions of posts and quality of iron, numerous tests had to be made before the material furnished by the four bridge companies, among whom the contracts for our iron structures are divided, could be accepted, and the sizes of posts determined upon.

"Tables Nos. 4, 5, 6, and 7, contain, in the order of date of tests, the results of experiments made on wrought-iron columns and struts of various kinds prepared by the four bridge companies. More of these tests were made than would have been necessary if the formula adopted in the specifications for the determination of the sectional areas of columns had been of universal authority, but though the best yet proposed, doubts were expressed by competent bridge-builders and engineers as to its correctness when applied to columns of certain shapes, and it was thought best to make a number of experiments sufficient to test the formula itself.

"For convenience of comparison the results for columns of the same kind have been put together in Tables Nos. 12 and 13. These tables also give the value of constant f, calculated for each column, by Gordon's formula, and also by the formula given by Rankine, in which the radius of gyration of the section is used in place of the smallest diameter.

"Gordon's formula . . . . $\dfrac{P}{S} = \dfrac{f}{1 + \dfrac{a\,l^2}{h^2}}$

"Rankine's formula . . . . $\dfrac{P}{S} = \dfrac{f}{1 + \dfrac{a'\,l^2}{r^2}}$

"P being the ultimate load producing the crushing or bending of column.

"S sectional area of column in square inches.

"f constant, supposed to be equal to the ultimate resistance per square inch of a short column, whose length is equal to its diameter.

a constant $\begin{cases} \text{For columns with flat bearings . . . .} & = \frac{1}{3000} \\ \text{For columns with flat bearing at one end and rounded at the other . . . .} & = \frac{2}{3000} \end{cases}$

a' constant $\begin{cases} \text{For columns with flat bearings . . . .} & = \frac{1}{36000} \\ \text{For columns with flat bearing at one end and rounded at the other . . . .} & = \frac{2}{36000} \end{cases}$

"l, length of column in inches.

"h, diameter of column in the direction of its greatest deflection.

"r, radius of gyration of cross-section of columns in the direction of its greatest deflection.

"In order to test thoroughly the mathematical correctness of the formula, ex-

periments should have been made with the same pressure on columns of different lengths and shapes of cross-section, made of the same iron, of uniform quality, and all fittings made, and measurements taken with great precision. All these conditions could not be realised. But for the objects in view—which were to ascertain whether the formula could be applied in practice with very approximately correct results, and, if so, to determine the correct value of the constant f—scientific nicety was not necessary; it was preferable, on the contrary, that the conditions of the experiments should be the same as those actually met with in posts and struts as they stand in iron structures.

"The following general conclusions can be drawn from the examination and comparison of all the tests made on columns:—

"1. For columns of the same shape, of different lengths, made of the same kind of iron, the values of f calculated from the formulæ, do not differ more than can reasonably be accounted for by the ordinary want of unformity in the quality of the iron, the differences not being greater than those between the ultimate tensile strengths, obtained with specimens of iron of the same manufacture.

"2. For columns of different shapes of cross-section, and made of different kinds of iron, the values of f calculated do not vary more than does the strength of iron of different manufactures.[1]

"3. Gordon's and Rankine's formulæ must both be considered as practically correct, for columns with flat bearings at the ends, of different lengths and shapes of cross-section, provided the ultimate strength of one column, made of the iron to be used be determined, so that the value of the constant f to be applied in the formulæ may be calculated. This constant being very approximately proportional, but not equal to the ultimate strength of the iron, and not the same in the two formulæ, being smaller for Rankine's than for Gordon's.

"4. For columns hinged on pins at the ends, Gordon's and Rankine's formulæ for columns rounded at one end and fixed at the other, will give approximately correct results, provided f be determined as noted above.

"5. For swelled columns, the formulæ are also applicable, provided the diameter at the end be used.

"6. It is of great importance for all built columns, and especially for open columns, that the several parts should be well riveted at the ends, and that true and even bearings at the ends be obtained.

"In determining the sectional areas of the columns and struts for each structure, the foregoing conclusions were followed.

"Tests Nos. 6 and 12 confirmed the fact already pointed out by some experimenter, but not universally admitted, that wrought-iron, when submitted to a pressure beyond that corresponding to its limit of elasticity, and allowed to rest, will afterwards possess a higher limit of elasticity.

"The singular results given by tests 13 and 19, in which both columns bent in the direction giving the greatest radius of gyration, which for No. 13 is perpendicular to the plane of rotation around the end pin, can be explained by defective fittings at the ends, inadequate balancing of the columns in the centre, or insufficient riveting, allowing a slight longitudinal motion of the two channel bars in opposite directions."

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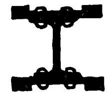
[1] Columns should be tested in a vertical, and not in a horizontal position as these were. But if tested horizontally, each column should be counterweighted with a weight equal to half its own weight, attached by a chain to the centre, and passing over a pulley. This was done in all the columns but the Phœnix columns, and had they been counterbalanced they would have given considerably higher results than those attained.—T. C. C.

AMERICAN BRIDGE CO.'S COLUMNS. TESTS MADE at CHICAGO.



No.	Kind.	Diameter.	Length.	Ratio of Length to Diameter.	Sectional Area.	Square of Radius of Gyration. (Unit = 1 inch.	Limit of Elasticity.	Resistance per square inch = $\frac{\text{"P"}}{S}$.	Modulus of Elasticity.	"f" by Gordon Formula.	"f" by Rankine Formula.	Remarks.
		Inches.	Ft. Ins.		Sq. Ins.		Lbs.	Lbs.				
12	Flat ends	$10 \times 9\frac{1}{2}$	20 0	25·3	20·10	8·653	15,000	..	..	..	..	Uneven bearings at ends; test discontinued after a pressure of 19,000 lbs. per square inch had been obtained.
18	,,	$10 \times 9\frac{1}{2}$[1]	20 0	25·3	20·10	8·653	23,000	31,500	23,600,000	37,500	37,300	Same column as above tested second time. Bent sideways.
19	,,	$10 \times 9\frac{1}{2}$	27 0	34·1 (32·4)	20·10	13·510 (8·635)	24,000	27,800	32,900,000	38,600 (37,500)	33,700 (37,200)	Bent upward. The numbers between parentheses apply to the column bending sideways.
15	,,	$8\frac{3}{8} \times 8$[1]	30 0	45·0	14·97	5·388	13,000	23,700	26,000,000	39,700	39,500	Bent sideways.
16	Hinged ends	8×8[1]	20 0	30·0	12·50	5·479	15,000	26,700	28,900,000	42,700	42,300	,, ,,
17	,,	$10\frac{1}{2} \times 10$[1]	20 0	24·0	19·90	8·733	12,000	26,500	23,100,000	36,700	36,200	,, ,,
13	,,	$12 \times 10\frac{3}{4}$[1]	26 0	29·0	25·05	18·215	12,000	24,000	30,400,000	37,500	31,100	Bent downward.
14	,,	$10\frac{1}{2} \times 10$[1]	26 0	31·2	20·72	8·733	14,000	22,000	26,000,000	36,300	35,600	Bent sideways.

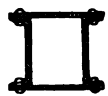
[1] Diameters used in obtaining the value of "f" by Gordon formula.

Hinged ends in above table mean pin bearings.

PHŒNIX COLUMNS. TESTS MADE  at CHICAGO and PITTSBURG.

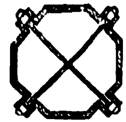
No.	Kind.	Diameter.	Length.	Ratio of Length to Diameter.	Sectional Area.	Square of Radius of Gyration. (Unit = 1 inch.	Limit of Elasticity.	Resistance per square inch = "P"/S.	Modulus of Elasticity.	"f" by Gordon Formula.	"f" by Rankine Formula.	Remarks.
		Inches.	Ft. Ins.		Sq. Ins.		Lbs.	Lbs.				
6	Flat ends	8·05	15 0	22·4	14·09	8·536	35,000	37,500	27,400,000	43,700	41,500	Had been compressed previously.
10	"	8⅓	27 0	39·9	13·70	8·935	18,000	31,000	29,100,000	47,400	41,100	Ends riveted closely.
28	"	8¼	28 0	40·7	13·58	8·935	22,000	34,800	25,700,000	54,600	47,000	" " "
29	"	8¼	28 0	40·7	13·58	8·935	18,000	36,600	28,500,000	57,500	49,400	" " "
11	Ends rounded with large radius	8⅓	27 0	39·9	13·89	8·935	17,000	21,700	27,100,000	44,700	35,900	By formulæ for hinged ends.
										67,700	50,000	By formulæ for round ends.

Hinged ends in above table mean pin bearings.

SQUARE COLUMNS. TESTS MADE  at CHICAGO and PITTSBURG.

No.	Kind.	Diameter.	Length.	Ratio of Length to Diameter.	Sectional Area.	Square of Radius of Gyration.	Limit of Elasticity.	Resistance per square inch.	Modulus of Elasticity.	"f" by Gordon Formula.	"f" by Rankine Formula.	Remarks.
23	Flat ends	10 × 8 7/16 [1]	24 0	34·1	13·70	11·628	15,000	33,200	28,900,000	46,100	39,800	Baltimore Bridge Co. Bent downward.
22	"	10 × 7½ [1]	26 0	41·6	13·60	9·347	16,000	30,000	27,800,000	47,300	38,700	Louisville Bridge Co. Bent upward.
32	"	10½ × 9¼ [1]	27 0	30·9	26·05	10·909	15,000	30,200	30,100,000	39,800	38,300	Keystone Bridge Co. Bent sideways.
21	Hinged ends	10 × 7½	25 9	30·9	13·60	11·000	18,000	25,500	31,000,000	41,700	37,800	Louisville Bridge Co. Bent sideways.

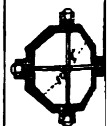
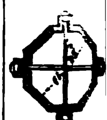
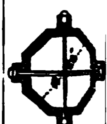
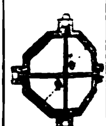
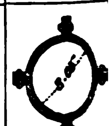
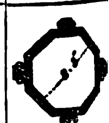
[1] Diameter used in obtaining "f" by Gordon formula.

KEYSTONE COLUMNS.  TESTS MADE at PITTSBURG.

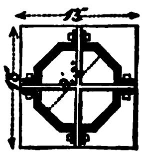
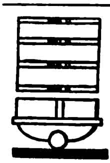
No.	Kind.	Diameter.	Length.		Ratio of Length to Diameter.	Sectional Area.	Square of Radius of Gyration. (Unit = 1 inch.)	Limit of Elasticity.	Resistance per square inch = $\frac{\text{“P”}}{\text{S}}$.	Modulus of Elasticity.	“f” by Gordon Formula.	“f” by Rankine Formula.	Remarks.
		Inches.	Ft.	Ins.		Sq. In.		Lbs.	Lbs.				
1	Flat ends, closed	$8\frac{1}{8}$	0	9	1·1	14·25	..	..	45,000 [a] / 51,500 [b]	..	..	..	([a]) Signs of buckling. ([b]) Crushing fast.
7	”	8·3	15	0	21·7	14·62	9·206	17,500	30,000	23,800,000	34,700	32,900	
9	”	8·85	15	0	20·3	23·67	7·833	19,000	32,000	24,700,000	36,400	35,700	Cross plates between flanges.
27	”	$8\frac{5}{8}$	27	0	37·6	18·83	9·798	18,000	27,800	23,700,000	40,800	36,000	
24	Flat ends, open, straight	$9\frac{1}{8}$	27	0	34·1	19·20	12·041	15,000	25,000	26,500,000	34,700	31,100	Riveted diametrically.
26	”	$9\frac{3}{8}$	27	0	34·6	14·49	11·178	17,000	27,500	27,500,000	38,400	34,700	
30	”	$9\frac{1}{8}$	27	0	34·1	15·13	11·464	12,000	30,000	19,300,000	42,100	37,600	Riveted at ends 18 ins.
2	Flat ends, open, swelled	9·3 × 10·1[1]	5	0	6·5	14·25	11·044	..	33,600	..	34,100	33,900	Castings at ends.
3	”	$9\frac{1}{4}$ × $11\frac{1}{4}$[1]	15	0	19·5	14·84	10·834	..	28,800	34,600,000	32,400	31,200	” ” ”
8	”	9 × $11\frac{1}{2}$	15	0	20·0	14·80	10·353	15,000	36,900	29,600,000	41,800	40,100	Buckled between bolts. Riveted at ends 12 ins.
4	”	9·2 × 11·62[1]	27	0	35·2	12·96	10·883	16,700	24,100	..	34,100	30,600	Castings at ends.
25	”	$9\frac{5}{8}$ × 12[1]	27	0	33·7	18·83	11·424	12,000	21,100	28,100,000	29,100	26,500	” ” ”
31	”	$9\frac{1}{8}$ × 12[1]	27	0	34·1	15·13	11·464	16,000	25,400	23,600,000	36,100	31,900	Riveted at ends 18 ins.
5	Hinged ends, open, swelled	9·22 × 11·6[1]	27	0	35·1	13·12	10·945	15,000	22,000	29,500,000	40,100	33,700	Bolted to castings at ends.

[1] Diameter at centre of Columns.

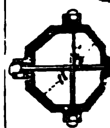
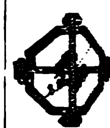
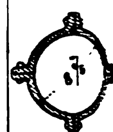
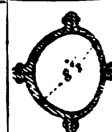
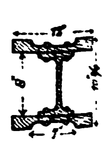
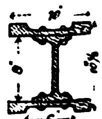
Experiments on Wrought-Iron Columns.

No.	Kind of Column.	Iron, where Rolled.	Tests where Made.	Date of Test.	Length of Column.	Sectional Area in Square Inches.	Middle Sect and Diameter.
					Feet. Ins.		
1	Keystone, riveted through flanges.	Union Iron Mills, Pittsburg, Penn.	Pittsburg, Penn.	April 19th, 1875.	0 9	14·25	
2	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 19th, 1875.	5 0	14·25	
3	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 19th, 1875.	15 0	14·84	
4	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 20th, 1875.	27 0	12·96	
5	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 21st, 1875.	27 0	13·12	
6	Phœnix, riveted through flanges.	Phœnixville, Penn.	Do.	April 27th, 1875.	15 0	14·09	
7	Keystone, riveted through flanges.	Union Iron Mills, Pittsburg, Penn.	Do.	April 27th, 1875.	15 0	14·62	

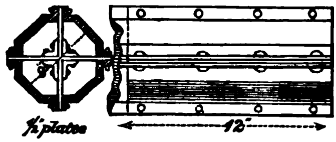
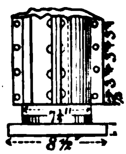
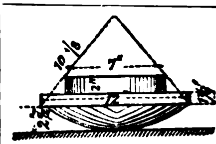
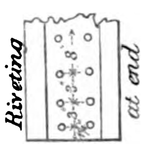
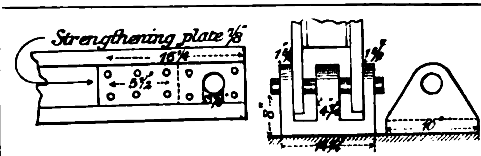
EXPERIMENTS ON WROUGHT-IRON COLUMNS.

tance om tre to ntre ivets.	Pressure per Square Inch producing Permanent Set, in lbs.	Pressure per Square Inch causing Fracture, in lbs.	Modulus of Elasticity.	Remarks.
hes. 5	40,000 Commenced buckling between rivets.	51,500 Plates buckled.	..	Planed at both ends.
5	..	33,600 Broke by buckling between rivets.	..	 Fitted with square castings and bolted at ends.
5	..	28,800 Broke by deflection.	..	Same as above. Diameter 9·25 inches at ends.
5	16,700	24,100 Broke by deflection.	..	Same as above. Diameter 9·2 inches at ends.
5	15,000	22,000 Broke by deflection.	29,500,000	 Square castings at ends with flanges fitting on a pin, the whole length of the latter bearing against an abutting casting. Diameter of column at ends = 9·22 inches.
3	35,000	37,500 Broke by deflection.	27,400,000	Column planed at both ends. No shoes.
3	17,500	30,000 Broke by deflection.	23,800,000	Column planed at both ends. No shoes.

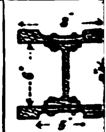
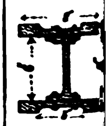
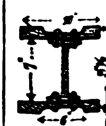
EXPERIMENTS ON WROUGHT-IRON COLUMNS—*continued.*

No.	Kind of Column.	Iron, where Rolled.	Tests, where Made.	Date of Test.	Length of Column. Feet. Ins.	Sectional Area in Square Inches.	Middle S… and Diamet…
8	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 28th, 1875.	15 0	14·8	
9	Keystone, riveted through flanges. Column not swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	April 28th, 1875.	15 0	23·67	
10	Phœnix, riveted through flanges.	Phœnixville, Penn.	Chicago, Ill.	June 2nd, 1875.	27 0	13·7	
11	Phœnix, riveted through flanges.	Phœnixville, Penn.	Do.	June 2nd, 1875.	27 0	13·89	
12	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 2nd, 1875.	20 0	20·1	
13	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 2nd, 1875.	26 0	25·05	
14	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	26 0	20·72	

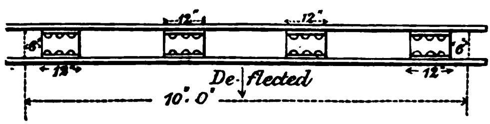
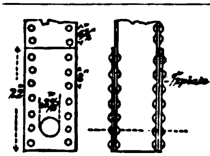
EXPERIMENTS ON WROUGHT-IRON COLUMNS—*continued.*

Pressure per Square Inch producing Permanent Set, in lbs.	Pressure per Square Inch causing Fracture, in lbs.	Modulus of Elasticity.	Remarks.
15,000	36,900 Broke by buckling between rivets.	29,600,000	 Ends riveted with cross plates and angle irons, and planed.
19,000	32,000 Broke by deflection.	24,700,000	Column planed at both ends. No shoes.
18,000	31,000 Broke by deflection.	29,100,000	 Column fitted with square castings at ends, and last four rivets spaced 3 inches from centre to centre.
17,000	21,700 Broke by deflection.	27,100,000	 Ends closely riveted as above and fitted with castings having spherical abutting surfaces.
15,000	Column discovered not to fit well, and trial discontinued at 19,000.	20,200,000	 Ends planed to fit squarely on abutting casting.
12,000	24,000 Broke by deflection downward.	30,400,000	 Pin ends.
14,000	22,000 Broke by deflection sideways.	26,000,000	Pin ends as above.

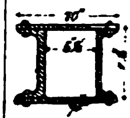
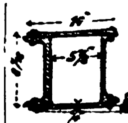
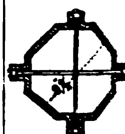
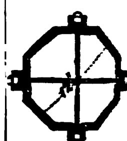
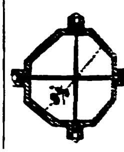
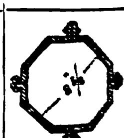
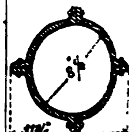
EXPERIMENTS ON WROUGHT IRON COLUMNS—*continued.*

No.	Kind of Column.	Iron, where Rolled.	Tests, where Made.	Date of Test.	Length of Column.		Sectional Area in Square Inches.	Middle Section and Diameter.
					Feet.	Ins.		
15	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Chicago, Ill.	June 3rd, 1875.	30	0	14·97	
16	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	20	0	12·5	
17	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	20	0	19·9	
18	American Bridge Co., same column as for No. 12.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	20	0	20·1	
19	American Bridge Co.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	27	0	20·1	
20	American Bridge Co.'s strut for girders of iron viaducts.	Cleveland Rolling Mills, Cleveland, O.	Do.	June 3rd, 1875.	10	0	9·66	
21	Louisville Bridge Co.'s square column: 2 channel bars and 2 plates, riveted.	Ohio Falls Iron Works, New Albany, Indiana.	Do.	June 29th, 1875.	25	9	13·6	

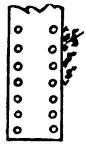
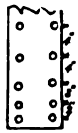
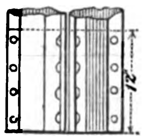
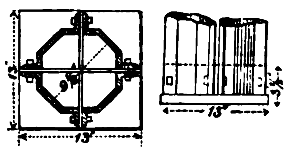
EXPERIMENTS ON WROUGHT-IRON COLUMNS—*continued.*

ance m re to re of ets.	Pressure per Square Inch producing Permanent Set, in lbs.	Pressure per Square Inch causing Fracture, in lbs.	Modulus of Elasticity.	Remarks.
hes. 8	13,000	23,700 Broke by deflection sideways.	26,000,000	Ends planed and riveted as for No. 12. Column balanced at centre with 500 lbs.
8	15,000	26,700 Broke by deflection sideways.	28,900,000	Pin ends as for No. 13.
8	12,000	26,500 Broke by deflection sideways.	23,100,000	Pin ends as for No. 13.
8	23,000	31,500 Broke by deflection.	23,600,000	Ends planed and riveted as for No. 12.
8	24,000	27,800 Broke by deflection upward.	32,900,000	Ends planed and riveted as for No. 12.
..	8,000	20,000 Broke by deflection downward.	21,600,000	 Pin ends as for No. 14.
4½	18,000	25,500 Broke by deflection sideways.	31,000,000	 Pin ends. Castings as for No. 14. Column balanced with 300 lbs. at centre.

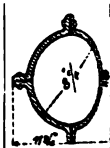
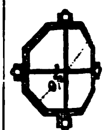
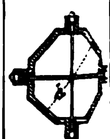
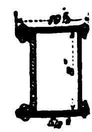
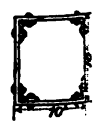
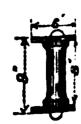
EXPERIMENTS on WROUGHT-IRON COLUMNS—*continued.*

No.	Kind of Column.	Iron where Rolled.	Tests where Made.	Date of Test.	Length of Column.	Sectional Area in Square Inches.	Middle Section and Diameter.
					Feet. Ins.		
22	Louisville Bridge Co.'s square column: 2 channel bars and 2 plates, riveted.	Ohio Falls Iron Works, New Albany, Indiana.	Chicago, Ill.	June 29th, 1875.	26 0	13·6	
23	Baltimore Bridge Co.'s square column: 2 channel bars and 2 plates, riveted.	Pencoyd Iron Works, near Philadelphia, Penn.	Do.	June 29th, 1875.	24 0	13·7	
24	Keystone, riveted diametrically between flanges. Column not swelled.	Union Iron Mills, Pittsburg, Penn.	Pittsburg, Penn.	July 2nd, 1875.	27 0	19·2	
25	Keystone, riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	July 3rd, 1875.	27 0	18·83	
26	Keystone, riveted diametrically between flanges. Column not swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	July 3rd, 1875.	27 0	14·49	
27	Keystone, riveted through flanges. Column not swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	July 3rd, 1875.	27 0	18·83	
28	Phœnix, riveted through flanges.	Phœnixville, Penn.	Do.	Aug. 5th, 1875.	28 0	13·58	

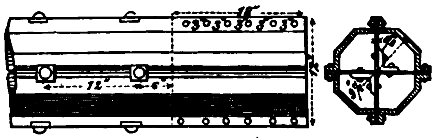
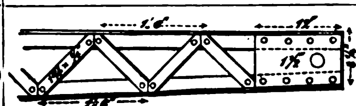
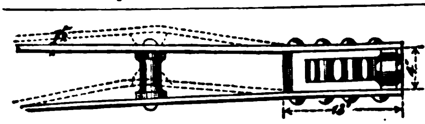
EXPERIMENTS ON WROUGHT-IRON COLUMNS—*continued.*

Distance from Centre to Centre of Rivets.	Pressure per Square Inch producing Permanent Set, in lbs.	Pressure per Square Inch causing Fracture, in lbs.	Modulus of Elasticity.	Remarks.
Inches.				
4½	20,000	30,000 Broke by deflection upward.	27,800,000	 Ends planed and closely riveted. Column balanced at centre with 300 lbs.
4	15,000	33,200 Broke by deflection downward.	28,900,000	 Column twisted before test $\frac{7}{16}$ inch. Ends planed and closely riveted. Plates buckled. Column balanced at centre with 300 lbs.
12	15,000	25,000 Broke by deflection sideways.	26,500,000	 Ends fitted with diagonal plates and planed.
12	12,000	21,100 Broke by deflection downward.	28,100,000	 Fitted with square castings and bolted at ends.
12	17,000	27,500 Broke by deflection downward.	27,500,000	Ends as for No. 25.
12	18,000	27,800 Broke by deflection downward.	23,700,000	Ends planed. Iron of column soft and badly welded in rolling.
6	22,000	34,800 Broke by deflection downward.	25,700,000	Ends planed. Last five rivets at both ends spaced 3 inches from centre to centre. Balanced at centre with 650 lbs.

Experiments on Wrought-Iron Columns—*continued.*

No.	Kind of Column.	Iron, where Rolled.	Tests, where Made.	Date of Test.	Length of Column.	Sectional Area in Square Inches.	Middle Section and Diameter.
					Feet. Ins.		
29	Phœnix, riveted through flanges.	Phœnixville, Penn.	Pittsburg, Penn.	Aug. 5th, 1875.	28 0	13·58	
30	Keystone, riveted diametrically between flanges. Column not swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	Aug. 5th, 1875.	27 0	15·13	
31	Keystone riveted diametrically between flanges. Column swelled.	Union Iron Mills, Pittsburg, Penn.	Do.	Aug. 5th, 1875.	27 0	15·13	
32	Keystone, square column, 2 channel bars and 2 plates, riveted.	Union Iron Mills, Pittsburg, Penn.	Do.	Aug. 26th, 1875.	27 0	26·05	
33	American Bridge Co.'s latticed strut for iron trestles.	Cleveland Rolling Mills, Cleveland, O.	Do.	Nov. 22nd, 1875.	28 6⅜	4 angle irons, 5·68	
34	Louisville Bridge and Iron Co.'s strut.	Ohio Falls Iron Works, New Albany, Indiana.	Chicago, Ill.	Dec. 4th, 1875.	12 3	6 00	

EXPERIMENTS ON WROUGHT-IRON COLUMNS—*continued.*

nce m e to e of ts.	Pressure per Square Inch producing Permanent Set, in lbs.	Pressure per Square Inch causing Fracture, in lbs.	Modulus of Elasticity.	Remarks.
es.				
6	(?) 18,000	36,600 Broke by deflection upward.	28,500,000	Ends planed and riveted as for No. 28. Column balanced at centre with 650 lbs.
2	12,000	30,000	19,300,000	 Ends planed. No shoes.
..	16,000	25,400	23,600,000	Ends as for No. 30.
6	15,000	30,200 Broke by deflection sideways.	30,100,000	Column planed at both ends. No shoes.
8	21,100	31,700 Broke by deflection upward.	32,400,000	 Angle-irons 2½ × 2½ inches. End plates ½ inch thick. Angle - irons fractured at one end near plates, and near middle of strut.
5½	16,000	17,600 Broke by deflection upward.	..	 Flat ends. Dotted lines show manner in which column gave way.

[Mr. W. H. BARLOW.

Mr. W. H. Barlow, Vice-President, said considering the long list of American girders of great span erected since 1870, exceeding 300 feet, and reaching 515 feet, he thought English engineers might look with envy upon their American brethren, in having such numerous opportunities for the exercise of their professional skill. In the Author's description of the method in which Mr. Linville had erected the bridge over the Ohio river, and of the manner in which Mr. Shaler Smith had erected the bridge over the Kentucky river, there were examples of facilities of resource in American engineers which both the older and younger members of the profession might follow with great advantage. In large spans it was a matter of considerable importance to study the details of the structure. There was undoubtedly a considerable difference in the use of what the Author called pin-connections as compared with rivet-connections. He had, to a certain extent, looked into that subject; and he had found that in suspension bridges there was a surplus of metal of about 10 per cent. arising from the use of pin-connections, while in riveted girders the waste amounted to 30 per cent., so that there was a large saving in the former method. It was to be observed, however, that a pin-structure, which was a joint, was liable to motion, and he was not quite sure whether in point of durability it was as good as the other. The sixteen bridges mentioned in the Author's tabular statement might be arranged under four classes: (1) what he had termed quadrangular bridges with pin-connections; (2) containing one bridge, Mr. Brunel's, at Saltash; (3) lattice bridges; and (4) tubular bridges. He had endeavoured to compare the efficiency and structural merit of those bridges by the system of ascertaining their limiting spans. It was a mode of treating the subject which he had adopted some years ago. It was first published by Professor Rankine, and it afforded great facility in ascertaining what the weight of a girder should be without the trouble of calculating all its several parts; and in like manner it afforded the means of computing how far one girder was superior to another. He meant by the limiting span the length to which a girder could be carried, increasing all its proportions in like manner, when its own weight would produce upon it the strains which were said to arise in the girder itself. Or it could be carried a little further, by ascertaining the length to which the girder might be carried so as to bring upon it those strains which English engineers were in the habit of putting upon girders—5 tons tension and 4 tons compression. With regard to the quadrangular girders mentioned in the list, the first of

them had a limiting span of 901 feet, and the others had limiting spans of 952 feet, 855 feet, 858 feet, 852 feet, and 982 feet, giving an average of 900 feet. The last girder in that group was number 16, and he desired to make a remark upon it. It was referred to in the Paper as being one of the best girders. It was one of Mr. Linville's girders, and it was evident that he was a clever constructor; but judging the girder in question by its limiting span, it did not appear to possess the same merit as the others. He thought, however, that that was not due to the girder itself, but to the manner in which the live load had been stated in the Paper, for whereas the live load of the other girders of Mr. Linville's had been stated at 3,000 lbs. to the foot, that was only put at 1,818 lbs., although in another part of the Paper it was referred to as 3,000 lbs. Taking it at 3,000 lbs. to the foot, that girder also came up nearly to the limiting span of 900 feet. He therefore came to the conclusion that the limiting span of girders of that construction was 900 feet. The next bridge, Mr. Brunel's, was of an early date, but in that case the limiting span was also 900 feet, or rather 1 per cent. above it. It was a bridge of very large height in proportion to its span. It might be said to be on the quadrangular system, inasmuch as the ties were at an angle of 45°, and it was pin-connected. It was known to many engineers that Mr. Brunel used in that bridge certain chains which had been previously made for a suspension bridge, and thence arose the pin-connections. This bridge undoubtedly ranked quite on a par with the best American bridges. In the next set, the lattice, it appeared that there was, as compared with the quadrangular system, a large waste of metal, apparently amounting to from 40 to 46 per cent. In the tubular bridges the result was still less favourable, and it appeared that nearly two-thirds of the weight might have been dispensed with; but it should be remembered that it was in tubular bridges that the first attempt was made to introduce wrought iron in large spans upon railways. That was done by the late Mr. Robert Stephenson at a time when perhaps it would not have been in the power of any other man to introduce wrought iron in such structures. He had, without exception, the best knowledge of mechanics and of structures of any man of his time; but that was thirty years ago, and great progress had since been made, which he believed was represented by the fact that the use of the structural knowledge possessed at the present time would have effected a saving of two-thirds of the metal used in those bridges. In making these remarks he relied upon the figures given in the Paper, and he was quite sure

that the Author, while undertaking the great labour of collecting the facts, had done his best to secure accuracy. But it was possible that some errors might be included, and it appeared to him that the "live load," stated to be carried by the girders should be further examined; because it was a material element in the calculations, and because in several cases the amounts put down under this head exceeded considerably the estimate of "live load" generally used in this country.[1] Before leaving this part of the subject it was proper to refer to the works of Sir John Macneill, who, as early as 1843, erected an open-work girder of wrought iron of 84-feet span. His principal work of that kind was the Boyne bridge, designed in 1851 and completed in 1855, under the superintendence of Mr. James Barton, M. Inst. C.E. It was a girder covering three spans, the centre opening being 264 feet, and the depth one-twelfth of the span. The bridge was open trellis work of great merit, in which full advantage was taken of the principle of continuity. In this bridge, if the transverse girders and horizontal bracing were taken as load, not contributing to vertical strength, the limiting span exceeded 800 feet.

With regard to the Author's observations upon steel, he did not think such careful consideration had been given to that material as to wrought iron. The days had gone by when it was possible to talk of the uncertainty of the strength of steel, it being used for all those purposes in which the greatest certainty of strength was required—wheel tires, rails, boilers, shafts, wire ropes for suspension bridges, ship-building, and the like. If it could be employed for these purposes why should it not be used for railway structures? The Author had spoken of it as being uncertain in its modulus of elasticity; at the same time he referred to the modulus of the elasticity of iron as varying from 20,000,000 to 28,000,000. He thought there must be something erroneous in the experiments upon iron. He had himself made many experiments upon iron, and he had never found so great variation in its modulus. The average was about 28,000,000; it varied only a little above and below that amount, and he thought there must be something wrong in the apparatus which gave so low a result as 20,000,000, unless it was wrought iron. Steel varied from 29,000,000 to 31,000,000 in its modulus of elasticity. He had made numerous experiments upon steel, and he could attest with-

[1] Since the Paper was read and discussed the Author has revised and made several corrections in the "Table of Tubular and Girder Bridges for Single-track Railways."—Sec. Inst. C.E.

out hesitation that in that respect it was one of the most reliable materials. There was a difference between elasticity and brittleness. The modulus of elasticity meant the degree to which a material would extend in proportion to its length by a given strain; and the modulus in steel, whether it was hard or soft, lay between the limits of 29,000,000 and 31,000,000. Steel had been so far recognised in this country that the Government had authorised its use in railway structures, assigning it a co-efficient of strength of 6½ tons to the square inch, that of iron being 5 tons; in other words, steel was regarded as a material possessing 30 per cent. more strength than iron. The Admiralty had made the same allowance for shipping, giving steel the same superiority of strength of 30 per cent. The Dutch engineers also credited steel with a coefficient 30 per cent. higher than iron. Applying to steel the question of the limiting span the 900 feet at once rose to 1,200 feet, and then the reduction of the weight of the material became considerable. But he did not suppose that steel would realise that full difference of 30 per cent. with regard to the limiting span. Those parts which were in compression would probably require more material about them to prevent buckling, when they were subjected to a strain of 6½ tons, than if they were of iron subjected to a strain of 5 tons. He thought it quite probable, that a limiting span of 1,150 feet would be realised. Assuming that to be so, the use of steel was of great importance in large spans, say above 600 feet. At 600 feet there would be as great a difference in the weight of the girder by the use of steel as compared with iron, as existed between the new American and the old-fashioned English construction. That was a matter of so much importance that he hoped it would be taken up by the Author. He observed that the Paper made no reference to archiform bridges, like the St. Louis bridge, with three arches of steel, and a bridge in Spain of 600 feet span, nor to several large suspension bridges. He understood, however, that the Author had not included those structures, as it would have been difficult to comprise them all in one Paper.

Mr. Benjamin Baker said the Institution was fortunate in having a Paper of that kind from so distinguished a bridge builder as Mr. Clarke. For some reason or other, the American type of structure had not found favour in England or in Europe. About a fortnight ago, a continental builder had offered him some American bridges, and when he mentioned the difficulty of finding a market for them in this country because of the prejudice against the type, the builder replied that they had been offered all over Europe, without a purchaser being found. There seemed to be

no reason for this prejudice, except, perhaps, that girders of the American type required to be very carefully manufactured, especially in the eye-bars. Of course, if they had come from the Author's works, no one would hesitate to rely upon them; but a few bad welds would make an American bridge a very dangerous and treacherous structure. The Author had compared the Ohio bridge and the Kuilenburg bridge, much to the disadvantage of the latter. He thought the Author had unintentionally done injustice to the Dutch engineers. The strain of the Dutch bridge was stated to be $6\frac{1}{2}$ tons to the square inch. Of course, no government would admit that strain, and there was no doubt some mistake in the figures. Again, the rolling load was taken at 595 tons. It was a double-line bridge, and no continental government would consider that a satisfactory test for such a bridge of 492 feet span. That the test load was greater was proved by the deflections given in the Author's table. As 3·05 inches (the deflection under the dead load) : 2890 tons : : 1·32 inch (the deflection under the live load) : 1250 tons, and not 595 tons as stated in the Paper. The Author appeared, therefore, to have under-estimated the rolling load by one-half. With that correction the Kuilenburg bridge, including the platform, carried about two and a half times the weight of the Ohio bridge, and the ironwork weighed barely twice as much; so that after all the Dutch bridge was not so much behind the American. He thought that since Mr. Colburn's Paper had been read,[1] there had been a great assimilation between American and European practice, and in no point more than in the strains. Mr. Shaler Smith had sent him a specification of a recent bridge, and he observed that in the floor suspenders there was a strain of only 2 tons to the square inch, whereas in some of the early timber bridges on the Erie railroad the strain on the suspenders was 23 tons, and even after the bolts had failed and been replaced by stronger ones, as much as 17 tons was imposed. On the other hand, European engineers had approached the American in doing away with stunted box-girders and tubular bridges, and in adopting the open lattice type of bridge. The difference was now mainly between pins and rivet work. The Author had claimed a number of advantages for pins, some of which he would allow, while others he would dispute. He said in one place that they were preferable on account of the mathematical certainty with which the strain could be calculated; but at the end of the Paper he stated that the limit of error in the links was

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxii., p. 540.

$\frac{1}{64}$ inch. Now an error of $\frac{1}{64}$ inch in a 10-foot eye-bar would mean a possible error in the strains to the extent of $1\frac{1}{2}$ ton to the square inch. If, in a given group of eye-bars constituting the tension member of a quadrangular girder, the strain might vary + and − $1\frac{1}{2}$ ton per inch, he was quite prepared to admit the result was near enough for practical purposes, but he should not term it "mathematical certainty" of strain. In rivet work $\frac{1}{64}$ or $\frac{5}{64}$ inch error would have no effect at all upon the strains. In the discussion on M. Gaudard's Paper he had stated his opinion that rivets did not act as pins, closely fitting the holes, but rather as bolts gripping the plates together.[1] He had also stated that, as far as he was aware, no girders had been tested to destruction in which more than three plates were piled one on the top of another in the flanges, and he had said he should like to know what the ultimate resistance of such girders would be. Since that time he had had a couple of girders made, in order to decide whether in practice the rivets simply gripped the plates or whether it was really, as usually assumed, a question of shearing and bearing area. The girders were each of 20 feet span, 2 feet deep, with webs $\frac{1}{2}$ inch thick; four angle-irons, $2\frac{1}{2}$ inches by $\frac{1}{2}$ inch, and flanges generally 8 inches wide. In one girder the top and bottom flanges were alike, five plates $\frac{1}{2}$ inch thick; and in the other girder, the bottom flange had eight plates $\frac{1}{2}$ inch thick and $5\frac{1}{2}$ inches wide. He had tested them both, in order to see what the result would be, and found that it was just as he had anticipated; for up to a calculated strain of 14 tons per square inch in tension, there was no movement of the rivets. That they did not shift in the least, but gripped the plates together like so many bolts, or almost like a weld, was proved by the deflections and sets under successive strains which were relatively identical with those in solid rolled girders. If the rivets had acted as pins, offering a certain shearing resistance and demanding a certain bearing area, the deflections and sets would have been many times those observed, since the girders were of ordinary manufacture with punched holes and exhibited the usual irregularities in the spacing of the holes. The plates were reduced in number towards the ends for a central load, and the calculated shearing strain per square inch upon the rivets connecting the angle-irons with the flanges was identical with the tensile strain upon the latter. With a tensile strain of 14 tons, and a compressive strain of 10 tons per square inch, the permanent set was but 0·15 inch in

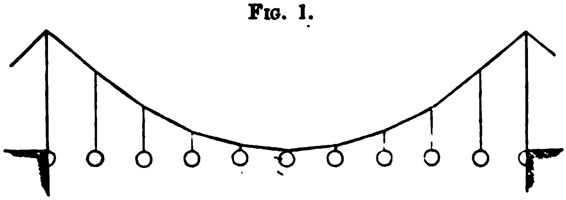
[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxxi., p. 152.

the five-plate, and 0·22 inch in the eight-plate girder, quantities in each instance so small as to preclude the possibility of the rivets having shifted and taken a bearing in the irregular holes in the flange plates, for rolled joists and even steel rails took as proportionally great a set. He had invariably found that badly punched girders with the holes partly blind, and the rivets tight but not filling the holes, deflected neither more nor less than the most accurately-drilled work, but he had never before carried on the test to the failing point in girders with as many as five plates and eight plates. He was rather surprised that the girder with eight plates failed by lateral flexure of the top flange under a calculated pressure of a fraction over 10 tons to the square inch. That seemed very low, because he had often tested rolled joists with flanges only 6 inches wide at the same span of 20 feet and they stood a strain of 0·18 ton to the square inch, or nearly double that of the wider riveted girder. In the case of the eight-plate girder at high strains, it became apparent that the plate nearest the angle-iron was doing the most work, though theoretically it should do the least, and probably the same was true of the five-plate girder. It would appear, therefore, to be unscientific and misleading to go into any great refinements in calculating the moment of inertia of an ordinary riveted girder, and it was clearly illogical, though convenient, to assume that rivets acted as pins when they really acted as clamps. The unsatisfactory character of the usual kind of girder calculations was not lessened by ignoring the fact of the great range in the modulus of elasticity. The Author referred to a case where the range was from 20,000,000 to 28,000,000; but Mr. Bender, who had occasion to test "many thousands" of eye-bars, found the moduli of bars up to 40 feet length, and from 1 to 14 square inches in area, vary from 18,000,000 to 40,000,000 lbs., and many other no less competent and scientific observers had noted great variations in the moduli of iron and steel. With regard to long-span bridges the Author had stated that under no circumstances could the girder on the cantilever system be less costly than a suspension bridge with a stiffening girder. He was under the impression that in the "Long Island" (New York) bridge competition the cantilever bridge with a central girder carried off the prize; and that the design condemned by the Author in the case of the bridge of 734 feet span, came out in the competition lighter, and a lower tender was made for its construction, than in the case either of the suspension bridge with a stiffening girder or the arch bridge. There might be some explanation of that, but he did not think that bridges should be compared, as the Author had done,

merely by taking the superstructure for certain distances on each side of the main span. Everything should be taken into consideration—foundations, anchorage, or abutments as the case might be. The cantilever bridge did not require the same anchorage as a suspension bridge; in fact, it did not necessarily require any anchorage at all, because it might be made to balance itself to a great extent. In the case of the arch bridge the iron for temporary use in erection ought to be included in the estimate. He should imagine that 400 or 500 tons would be required to erect the arch bridge. He had had occasion to design and estimate in detail several railway and other bridges of spans ranging from 600 to 1000 feet, and had found that no general rule could be laid down as to the most economical type of construction. Each case required to be considered individually with reference to the local and special conditions before any conclusion could be come to by the engineer as to the most generally advantageous type.

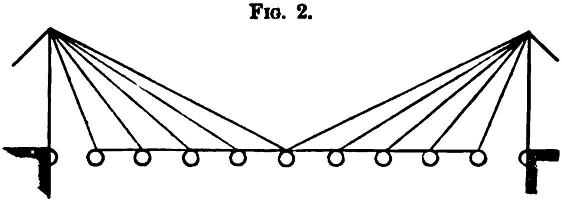
Mr. E. W. Young would confine himself to contesting the Author's statement that a stiffened suspension bridge was the cheapest for carrying a load over a long span; his object being to direct the discussion into the theoretical consideration as to what was the best method of carrying the load, because the improvement of details could not be of much avail if an expensive method of construction were adopted. Fig. 1 represented an

Fig. 1.



ordinary suspension bridge unstiffened, with an evenly distributed load. Calculating carefully the length of each bar in the structure with the strain upon it, multiplying the length of the bar by

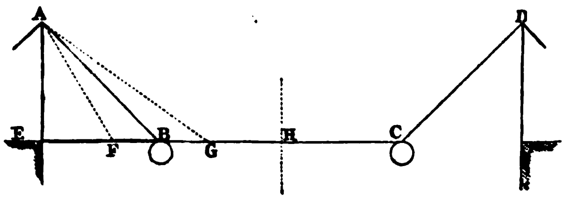
Fig. 2.



its sectional area or the strain upon it, which amounted to the same thing, and adding these quantities together, a sum was

obtained which might be taken to represent the quantity of material in the chains and the rods. If now the same load was carried by a system of inclined and horizontal tie-bars, as shown in Fig. 2, and a similar process was adopted as that applied to the system of Fig. 1, it would be found that the quantity of material in a structure of this form was exactly equal to that in a structure of the form of Fig. 1. Further, if the distributed load of Fig. 2 was concentrated in two places distant one-fourth of the span from the centre and carried by the bars A B, B C, and C D, as shown in Fig. 3, it would be found that the amount of material required

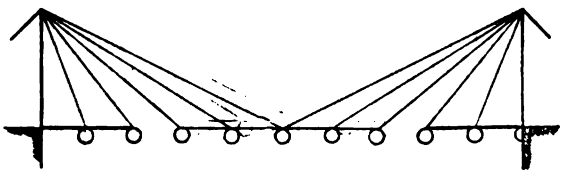
Fig. 3.



to carry the load concentrated in these positions was exactly equal to that required for the same load distributed and carried as in Fig. 2, and therefore, identical with that required for Fig. 1. Here he would observe that by constructing a diagram of the form of Fig. 3, and concentrating the load in the manner described, a ready means was afforded of computing the quantity of material in the chains and suspension rods of a suspension bridge, care being taken to add a proper percentage for eyes and other connections, thus saving the trouble of drawing out the whole bridge and calculating the strains in detail. Since the method of Fig. 2 was similar to that of Fig. 1, it was clear that if the load could be carried by a system cheaper than that of Fig. 2, the new system would be cheaper than that of Fig. 1, the common suspension bridge. Referring again to Fig. 3, it would be observed that the load at B was carried by the tension bars A B and B H, but the strut E B might be substituted for the tie-bar B H, and taking the same unit of strain per square inch on the strut as on the tie, the quantities in the cantilever A B, B E would be the same as those in the suspension bars A B, B H, since B E was equal in length to B H by construction. But if the load was placed closer to the pier, say, at point F, there would be a shorter strut E F, as against the tie-bar F H, and consequently to carry the load at point F it would be cheaper to use the cantilever than the suspension system, though the contrary would be the case for a load at G on the centre half of the span. So that for a load carried at any point between the

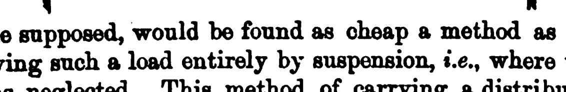
fourth of the span and the pier, it was cheaper to use the cantilever than the suspension method. For a distributed load the method shown in Fig. 4 would be more economical than that shown in Fig. 1. It was thus demonstrated that carrying by suspension was not the cheapest way of carrying a load between piers. He had not been taking into account the dead load of the structure itself; but if that were considered it would be seen that the cantilever system had a still further advantage over the suspension, because the weight of the bar B H, Fig. 3, had to be entirely borne on the point B, whereas the weight of the

FIG. 4.



strut E B was partly borne by the pier, only half of it coming on the point B. Then there was a further advantage in favour of the cantilever method when a live load had to be carried. The cantilever portion was rigid, while the suspension portion was yielding, and if any alteration took place in the loads, either in Fig. 2 or Fig. 1, the whole structure was deformed; therefore, when a moving load had to be carried it was necessary to introduce some method of resisting its distorting effect. Fig. 5 illustrated the carrying of a live load by suspension only.

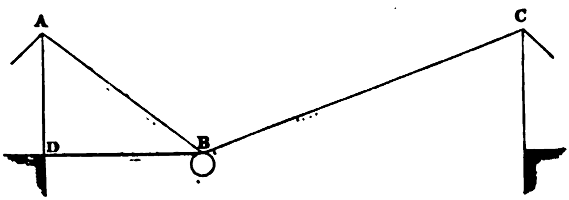
FIG. 5.



That, he supposed, would be found as cheap a method as possible of carrying such a load entirely by suspension, *i.e.*, where the dead load was neglected. This method of carrying a distributed load took about 50 per cent. more material than that of Fig. 1 or Fig. 2, so that it would be a mistake to apply it to the carrying of dead load. An economical rigid suspension bridge could, he considered, be obtained by a combination of the systems of Fig. 2 and Fig. 5, the former being used to carry the dead load, and the latter the live load only. Using the system of Fig. 5 it would

also be found that for portions of the load it was cheaper to use the cantilever system. In Fig. 6 he had shown a load suspended at B, one-third of the span from the pier, and he had found, by multiplying the sections of the material, or the length of the bars, by the strain, that precisely the same amount of material was required to carry the load at B by cantilever as by suspension; that was to say, the material in the bars A B, B C would be the same as the material in the bars A B, B D: therefore, if it were required to carry a live load so as to obtain rigidity, it would be cheaper to use the cantilever system for any load nearer the pier than one-third of the span; while on the other hand, it would be cheaper to carry the load by suspension when it was on the centre third of the span. There was, therefore, a point between a third and a fourth of the span in which a structure, to carry a load in the most economical form, should change from the cantilever to the suspension system; and the exact position of that point would depend, to a certain extent, upon the proportion

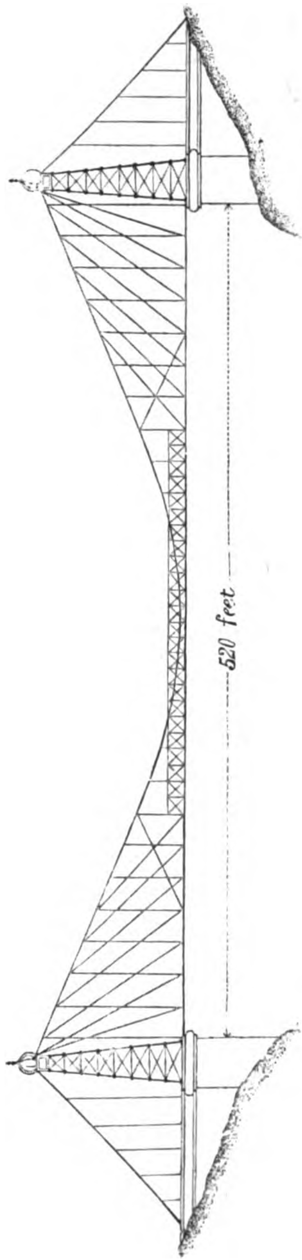
FIG. 6.



between the live and the dead load. The greater the proportion of the dead load to the live load, the nearer the point should be to the pier; the larger the proportion of the live load, the nearer should it be to the third of the span. For some years he had advocated that method of construction, and had consequently made many estimates for bridges of that kind, one of which was shown in Fig. 7 having a span of 520 feet. The result of his calculations had been to show that that system of construction was far more economical than that of a stiffened suspension bridge, because the stiffening girder formed a very serious item if it was to do its duty properly. He would mention two estimates that he had made with a view of comparing them with two bridges referred to by the Author. The bridge shown in Fig. 7, of 520 feet span, was really 540 feet between the bearings, and it was estimated to contain only 441 tons of material, including the towers, anchorages, and everything, as compared with 1,176 tons on the Ohio bridge, which was of 515 feet span. It was true he

had assumed that he was using steel, but that only compensated for the difference between the live loads in the two cases. Then he had made a comparison with Niagara bridge, which was a stiffened suspension bridge. To carry a live load of 500 tons it had 800 tons of iron between the piers; whereas a bridge which he had designed to carry 1,125 tons rolling load was estimated to weigh only 728 tons. The economy attained in these bridges was due not only to the mode of construction but to the height of tower. In most suspension bridges the tower was a great deal too low for economy. About $\frac{1}{8}$ of the span was the most economical height if a cheap class of tower were adopted. That, in a great measure, explained the difference in the weight; and he was therefore all the more astonished at the Author's figures. He had not given the chain the full benefit of the versed sine; he had not carried it to the bottom of the girder. He had also made the stiffened girder extremely shallow, $\frac{1}{38}$ of the span. It ought to be much less than that for economy; but even so he got less material in the stiffened suspension bridge than in the bridge on the cantilever system. With regard to the Eads arch, it was the converse of the suspension bridge. Instead of putting a stiffening girder he stiffened the arch. He did not think it was so economical, because it conveyed a transverse strain to the arch, which then became subject to complicated strains. He considered that in a bridge of this character it would be best to make the arch of the most suitable form to resist compression, and to connect it with a well proportioned

Fig. 7.



horizontal stiffening girder. Both arch and girders might be hinged at the centre of the span with safety. Again, means should be provided for preventing the hinge on the top of the pier from buckling outwards or inwards when the bridge was unequally loaded. If all that were taken into account it would materially increase the amount of material used in the Eads arch. It was quite possible to brace bridges, of the compound character to which he had referred, by making the platform a horizontal girder laid on its side having points of contrary flexure at the junction of the cantilever and suspension portions, and splaying out the cantilever portion towards the piers. It was easy to make them stiff enough to resist the wind with safety. In every bridge designed by him 40 lbs. pressure per square foot had been allowed, and security obtained.

Mr. G. H. PHIPPS thought it desirable to draw a distinction between the designs of bridges in relation to their scientific construction, as completed *in situ*, and to such modifications as might be necessary for facilitating their erection. As regarded the first condition, he scarcely thought that either in the Kentucky bridge, No. 7, or in the Ohio River bridge, No. 11, the construction was free from defect, as in both designs the introduction of vertical members between the booms was a wasteful use of material. When viewed, however, with regard to the remarkable manner of erecting the Kentucky bridge (by building out as a cantilever), it was perhaps necessary, and certainly convenient, to make the diagonal bars wholly tensile, and so to necessitate the use of the vertical pillars. In contemplating the economy of the above and other larger bridges described in the Paper, it would be noticed how much was due to the high proportion of depth to span in these girders :—

In the Britannia bridge, erected in 1850, the ratio was . .	$\frac{1}{16}$
Boyne bridge, completed in 1855	$\frac{1}{12}$
Kentucky bridge, completed in 1877	$\frac{1}{10}$

When the Britannia bridge was built, the proportion of 1 in 16 was adopted as the most economical with sides of its massive construction rendered enormously heavy by the great number of vertical pillars, then thought necessary. The Boyne bridge was a great improvement and an admirable construction. Its lighter lattice sides admitted of an increase in depth, while their open construction offered less lateral obstruction to the wind. The Kentucky bridge, however, went still farther in height. The Author spoke of the practice (now most usual in the United States) of constructing the booms of girders with assemblages of bars jointed

with pins like the chains of a suspension bridge. This had been done to a large extent in the lower booms by Mr. Berkley in India, and with great advantage for celerity of erection and facility of carriage. At first sight this manner of constructing the booms of girders appears to be economical in weight; but from his calculations it was only so when the pins were more than 11 feet apart. He had calculated the conditions of the lower boom of the Charing Cross bridge, where the bays were 11 feet, with the result that, calling the utilised metal 100, the additional metal in cover plates and metal rendered ineffective through effect of rivet holes, amounted together to 40 per cent. To contrast this with the chain-link system, he had taken the proportion of excess in the head ends over the general body of the bar, as shown in No. 4 example of Mr. Berkley's Paper "On the Strength of Iron and Steel,"[1] and he found that, when the length of bar was 4½ feet, the excess in the two heads was equal to the whole of the metal in the body of the bar; and that it was only when the length reached 11 feet that the percentage of losses on the two systems was about the same. This of course explained the necessity for economy of making the bars long (as mentioned by the Author); but when this was done, it led to a difficulty in the increased length, and consequent weight, of the rail-bearers, which was fixed by the distance asunder of the joints. On this and other points the want was felt of more detail of the construction of the American bridges. In reference to the building of very large bridges, he thought the best hopes were on the stiffened suspension bridge with the tensile portion of steel wire; and even in girder bridges he would throw out the suggestion to make the tensile boom of steel-wire cables.

Mr. Matheson said the depths of the girders referred to by Mr. Phipps were now generally regarded as obsolete; a better knowledge of the materials had led to the construction of deeper girders. He had been surprised to find how shallow were the girders described, in comparison with those supposed to be usually adopted in America. It had been imagined that a great deal of economy had been obtained in the American system by having the proportionate depth to the length of the girders as 1 in 7, or even less, and it was rather a concession to English practice that, in the bridges brought forward as good types, the proportion did not exceed 1 in 10. That the saving in weight which deep girders allowed was well understood in England might

[1] *Vide* Minutes of Proceedings, Inst. C.E., vol. xxx., p. 234.

be seen by Mr. Brunel's bridges at Chepstow and Saltash; but, then, it was also well known that the advantages were counterbalanced by disadvantages. The question of pins *versus* rivets was a point on which the English differed from the American bridge builders. He was glad to notice that the Author did not recommend pins for bridges under 100-feet span. There were numerous bridges in America of much less span made with pins—and many of them of what English builders would call a very loose style of construction. A few years ago pin-bridges were common in England, but he thought it was the opinion of the best engineers that, although for some years they might be as good as riveted bridges, and show a greater economy in construction, they were not permanently so good. The $\frac{1}{64}$ inch clearance, recommended by the Author, would, in his opinion, with the pins, show in time much deterioration. The pins would be liable to hammer under railway traffic, while rivets would form part of the absolute structure, and in fifty years would, as at the beginning, form part of the solid iron. With reference to the construction of links, a few years ago a firm in London rolled solid links without welds, and he considered them to be the best that were ever made; but that firm no longer existed, and no one else had taken up the manufacture. He believed one or two firms had begun to make links without welds, and it would be interesting to know how they were made. In America, by a combination of the hydraulic forging-press and steam-hammer, they were made quickly out of simple bars; but the Americans possessed some advantages not shared in England. English manufacturers had to obey the instructions of numerous engineers; no manufacturer could tell what design might be presented to him next week. In America the designer and the manufacturer were allied, and the manufacturer could afford to make thousands of links and lay them by, so that, when a bridge was wanted, it could be made in a few weeks. He did not say which system was the best; but, at any rate, it was a point that considerably affected the question under discussion. Economy could often be obtained by hauling over a girder, but a link-bridge girder could not be hauled over without special preparation, because the bottom member would collapse, while in a riveted girder it would temporarily form a strut. No doubt, however, in countries where there was no skilled labour, the facility with which pin-bridges could be put together on staging was a great advantage. The Author had compared the weight of American girders with the weight of English girders, to the disadvantage of the latter. No doubt the superior iron obtained in America allowed

economy in that direction; there was also a saving by leaving out certain portions, the omission of which would be forbidden in England. The Paper deprecated, for instance, such a waste of material as was involved in putting a platform on a bridge. That might explain the fact that trains in America often ran off the rails and fell through the bridge. He did not think any Government Inspector in England, Germany, or France would allow such a bridge to be constructed. A saving was also effected by omitting parapets, although such saving could hardly be put down to superiority of design.

Mr. C. Douglas Fox thought those members of the Institution who had had the privilege of visiting America from time to time, during the last twenty years, would realise the rapid progress that had been made there in iron construction generally, and towards the solution of the important question now brought forward by one of the most prominent American engineers in that particular line of business. In 1857, iron bridges were in their infancy in America; but since that time they had greatly increased in numbers, and exhibited a marked improvement in their details. He thought it was a matter of congratulation for English engineers that, as the Americans had increased their knowledge and experience, they had drawn nearer to the English types of construction. The practice in America of advertising, not only for tenders but for designs also, would, he thought, take many years to introduce into England; and although there might be advantages in it in some directions, on the whole it would certainly not tend to an increase of scientific knowledge, or to an improvement in scientific construction. With regard to the question of pin and rivet construction, he differed from some previous speakers as to the feeling of English engineers. In his own office it had been the practice for the last fifteen years to adopt pin construction for spans exceeding 80 feet; and below that span to retain the ordinary rivet construction and plate girders. There were many advantages in pin construction, especially for very large spans. The exact amount and direction of the strains at every point could be more readily calculated, and the strains could be carried at each connection by a single pin, the strength and bearing surface of which could be proportioned to the total strain at that point, instead of their depending upon the joint action, and perfect fit, of a number of rivets, which, as experience proved, often did not all come into use until some of them had been unduly strained. This was the case even where holes were drilled and every care taken to fill them. A

much worse result was naturally obtained where, as was too often the case, the holes were blind, or the rivets did not fill the holes. He must deprecate, as representing fairly the opinions or practice of English engineers, the view stated by a previous speaker, that such inferior workmanship made no practical difference in the strength or efficiency of riveted work. It had been said that that mode of construction was very defective, because dependent upon welds in the links. In all the bridges with which he had had to do of late years, he had never used a welded link for any important part of the structure. It was true that some years ago Messrs. Howard and Ravenhill were the sole manufacturers of rolled links; but there were at present several firms in this country who were able to produce a solid link by stamping out the eye by hydraulic pressure. A number of links were then placed together, and the holes at both ends drilled to template, so that they exactly corresponded, and the links were interchangeable. He had obtained links thus manufactured at a less price per ton than riveted work included in the same contract. There was no secret about the process; it was simple, and the result was satisfactory. But if the system were to be adopted to advantage, he had found it important that joints in the flanges at the pins should be avoided, and that no rivet should be introduced into the links. Many bridges were constructed where the links, although mainly connected by pins, had rivets introduced in order to connect the eye with the body of the link. Mr. Bruce and he had tried some experiments upon the subject, and they had been much struck by the remarkable results produced. They took links which had a sectional area of $6\frac{3}{4}$ inches by $\frac{7}{16}$ inch in the body, some stamped out of the solid, and others having their eyes riveted on, and tested them in Kirkaldy's machine, the result being that, while the rivets reduced the area only by 13 per cent., they reduced the strength of the link by 30 per cent.; and while before the insertion of the rivets the iron was able to carry a strain of 22 tons to the square inch, after the insertion of the rivets its power was reduced to $16\frac{1}{4}$ tons. With reference to the supposed difficulty of rolling over girders of that kind, he could only say that a bridge not far from London, of 120 feet span, had been thus treated under his own direction, temporary struts having been inserted to provide for the altered strains. Mention was made in the Paper of experiments to determine the exact proportions of links and pins. Mr. Bruce and he had tested a large number of full-sized links, pins, and struts, and he was interested to find that both these tests, and the latest American practice, very closely agreed with the results arrived at

twenty years ago by the late Mr. Vignoles, Past-President, and the late Sir Charles Fox, after an elaborate series of experiments for the purpose of determining the proportions of the Kieff Suspension Bridge,[1] though these results differed essentially from previous practice. With regard to the construction of bridges of large span, the comparative table given in the Paper did not appear to him to form a fair criterion. The ages of the English and American bridges were very different, and there was no mention of some particulars essential for forming a correct comparison of the various structures. It was not stated whether the rails were carried on the top or the bottom of the main girders, which made a great difference in the weight, nor were the relative proportions of the weights of the main and cross girders stated. Nor could the platforms of English bridges fairly be compared with those made on the system adopted in America, where engineers were not under such a strict *régime*. Then he thought that there must be some clerical error in the deflections given in the table, and that the permanent set must have been in some way mixed up with them. The Author had mentioned three types of construction as suggestions for railway bridges with large spans. Anything, however, that could be said on that subject could only be theory, because beyond 500- or 600-feet span—with the exception, perhaps, of the Niagara suspension bridge—there were no examples to go by. With reference to the arch proposed by Captain Eads, he believed that American engineers did not agree with him in considering that form of structure suitable for very large spans. In the first place, it would require an immense amount of bracing in order to keep it in shape; secondly, the workmanship, being of a curved kind and tubular, would be expensive; from the fact of its being tubular, it would be difficult to repair; and all the material in the arch was in compression, and therefore not used in the best possible manner. The suspension bridge was a much simpler structure, and it had the great advantage that all the material in it was in tension, and that the various parts of it took their fair share of the strain; it was, moreover, easy to erect. The difficulty with the suspension bridge for railway purposes was its want of rigidity. Those who had been over the Niagara suspension bridge had been no doubt struck with the boldness of the design; but when he crossed it he had felt thankful on reaching the other side—not from any doubt of the strength of the bridge, but from an uncomfortable feeling produced by the vibration and deflection.

[1] *Vide* Proceedings of the Royal Society of London, vol. xiv., p. 139.

He thought there was a great deal to be said in favour of the cantilever form of structure, and that, if progress was to be made in large structures, it would be made in some such direction as that. The subject had been discussed for many years, and various designs had been made. He had prepared a design for a railway bridge in Austria, in which the span was 1,350 feet, the depth of the valley being between 400 and 500 feet, so that piers were out of the question. The bridge was designed to be made of steel, and to be constructed, not exactly upon the cantilever principle explained by the Author, but something like it. The weight was a little under 1,600 tons between the towers, and the total weight a little over 2,000 tons. The rolling load was taken at 1 ton per lineal foot of span, and the maximum strains upon the steel were taken as 10 tons in tension, and 8 tons in compression. He thought it unnecessary to go to the very great weight shown in some of the designs in order to construct a satisfactory bridge. If engineers were to look forward to any rapid extension of bridges of large span, it should be in the direction of improved material, and their hopes in that direction were, at present, confined to the use of steel. On the general question he would say, first of all, avoid large spans altogether if possible, because, if a pier could be introduced, even though the cost were about the same, it would be a great advantage. A large bridge was no doubt a fine thing for the engineer who designed it, but it was expensive to construct and both costly and difficult to keep in order. The larger the span, the greater the risk in erection, and the greater the cost of maintenance. Then a cellular or tubular structure was objectionable, such as had been spoken of in connection with Captain Eads' arch, and one that could not be easily got at and painted. Engineers ought also to avoid anything like a continuous girder. He had designed many bridges upon the continuous principle, but had come to the conclusion that the complication of strain resulting from its adoption rendered it undesirable for plate girders, and especially objectionable for bridges with pin connections. He had been much struck by the ingenuity of the arrangement of the Kentucky bridge and the way in which it was rolled over; but it would have been all the better if the girders had been separated over the top of the piers instead of being hinged at intermediate points. It was most desirable to adopt sections which manufacturers could deal with without altering their machinery, and this it was quite possible to do. Another important point in dealing with large bridges, especially where there was no chance of getting an

intermediate support for the girders during repairs, was that all the parts should be made so that any one could be removed and replaced while the structure was still standing. In that way the bridge could be renewed without the immense expense of re-erection. Last, but not least, steel should be used, or some better material if it could be found. He believed that after a time manufacturers would be able in the ordinary way to produce a steel, such as could already be obtained by extra care, equal to a working load of 10 tons per square inch in tension and 8 tons in compression; and, when that was accomplished, there would be very little difficulty in building bridges with spans of 1,500 or 1,600 feet, at comparatively moderate cost.

Mr. MAX AM ENDE would give a few results of some calculations he had made on an *à priori* method of calculating the weights of girder bridges. He had applied the method to girder bridges of more than 500-feet span, and especially to four different kinds of girder: the straight girder with vertical struts and diagonal ties; the straight girder, with diagonal struts and diagonal ties; the parabolic bowstring girder; and the parabolic fish girder. He found that in the case of the first girder for 500-feet span the best proportion of depth to span was about $\frac{4}{10}$. It then increased, and became infinite at the limiting span, which was for iron of 5 tons strain per square inch about 2,870 feet. For the second girder the limiting span was about 4,000 feet for iron, and for steel of $7\frac{1}{2}$ tons strain 6,000 feet. The depth at the limiting span was infinite. The third girder had also a proportion of $\frac{4}{10}$ at first, and of $\frac{6}{10}$ at the limiting span of about 3,000 feet, the corresponding depth being 1,830 feet. The fish girder had a limiting span of 4,200 feet for iron and of 6,300 feet for steel, the corresponding depths being 3,600 feet and 5,400 feet. Those most economical depths were certainly much greater than had hitherto been assumed. But he found that the minimum of material was approached very gently, and that therefore, by reducing these depths considerably, the weight of the bridge would not be much increased.

Mr. G. WHITEHEAD, of the Hull Forge Company, as a manufacturer of bridge-links, should be glad to describe the mode in which they were made at his works. These works were constructed entirely for the manufacture of scrap iron. The old scrap was rolled into bars, which were cut up and formed into a pile sufficiently heavy to make a link of the full dimensions. After the pile was thoroughly hot and the ends of the link formed under the hammer, the pile was taken to the mill and rolled out; the ends were then cut, and there was the link in a substantial form.

The tests by Mr. Kirkaldy showed that the permanent set was 13 tons. After it was broken, it would reduce in area about 25 per cent. The process was simple, economical, and efficient. The holes were drilled after the links were cooled. It would be impossible to get them uniform in distance if they were punched. The iron could not be heated twice to regulate the distance, and he believed that the distance between the centres of the links was imperative. It was thought, by the engineers for whom they were making the links, that it was better and safer to drill the holes than to have them punched or hammered, or made by hydraulic pressure whilst hot.

Dr. SIEMENS remarked that it was stated in the Paper that the strength at which the iron was to be tested was equal to 60,000 lbs. (nearly 30 tons) to the square inch, and that it was to be subjected only to 10,000 lbs. of strain in tension. Surely there must be some error in that statement, as he knew of no iron that would stand a breaking weight of 60,000 lbs., except puddled steel, which was not the material used according to the specification, and if such iron could be obtained the weight with which it was to be loaded, viz., 10,000 lbs., appeared needlessly small, being equal to $\frac{1}{6}$ only of the total breaking weight. American engineers, if the Author represented them correctly, had very little confidence in steel as a building material. Perhaps he might be allowed to make a few remarks with regard to that material, to which he had paid considerable attention. A distinction ought to be drawn between steel and steel. There was a material called steel, but which, in reality, was the purest iron ever introduced to the notice of engineers. It contained 99·6 per cent. of metallic iron, and only 0·4 per cent. of foreign matter of every description, whereas the best so-called iron contained between 3 and 4 per cent. of foreign material. Mild steel was really iron of the best character, and he could not conceive how such a material could be thought unreliable in its application. It was produced, not like puddled iron, in small quantities to be welded together with the chance of enclosing foreign matter, and producing irregular results; but it was produced in large masses—10 or 12 tons of fluid substance—and there was every probability that such a material was uniform to the utmost degree. Practice had, indeed, fully substantiated the fact that there was no material more uniform than that very mild steel. It would bear 28 tons breaking strain to the square inch, simply because it was iron, for no one would pretend that cinder would bear a strain equal to that of iron. In mixing between 3 and

4 per cent. of cinder with the iron, some of its strength must necessarily be lost; otherwise it had all the qualities which iron ought to possess. But for engineering purposes he would restrict the use of very mild steel. It was excellent for the construction of boilers, and for the construction, perhaps, of continuous girders, because it was very reliable. It might be loaded to one-half its breaking strain without any sensible permanent set, and if it were loaded beyond that, it would not rupture, but it would elongate to the extent of 25 per cent.; therefore, in constructing a girder bridge of such material, the chances were that it would bend down to the bottom of the river rather than break. But it was not the strongest material that could be used. In the New York bridge, which he saw in progress last year, the steel wire was tested to nearly 100 tons per square inch. In his own practice he generally applied 80 or 90 tons as the weight which steel wire, for telegraphic purposes, ought to bear. But between the two limits—between the material that would bear a breaking strain of 28 tons, and elongate 25 or 30 per cent. before breaking, and the highly wrought material which would bear 80 or 90 tons to the square inch—there were a great many steps, all of which were applicable under circumstances such as practice must indicate. For links, he should consider that a material capable of bearing from 45 to 50 tons to the square inch would be the best material. It would be sufficiently yielding to excessive strain; it would yield 10 per cent. before rupture took place, and at the same time it could be loaded to certainly 15 tons to the square inch with safety. In constructing long bridges, the difference was so great, whether using a material that could be loaded safely and practically with 15 tons, or a material safely bearing a strain of only 5 tons, which was the limit now imposed upon iron, that there could be no question, in the long run, which was the right material for such large structures as were referred to in the Paper. As regarded compressive strain, steel must be looked upon as potentiated wrought iron, bearing a strain of compression equal to that of extension; but it must be borne in mind that the smaller scantling of the steel under compression necessitated a change in construction with a view to giving the requisite amount of lateral support; it was therefore not sufficient to make a design in iron and reduce the scantlings in substituting steel, as had been the practice to some extent.

Mr. Clarke, in reply, said he entirely concurred in the remarks of Dr. Siemens, except in regard to the doubt he had expressed whether American iron would bear 60,000 lbs. strain per square inch.

It was certainly rather an unusual specification. American engineers were generally contented with from 50,000 to 55,000 lbs.; but in the case in question, the specification absolutely called for 60,000 lbs., and experiments proved that this strength had been obtained. Reference had been made to a 6,000-feet span, with a height of 1,800 feet. But those were purely theoretical figures, and they took into account simply the forces of gravity. A bridge was a complex structure. It had to bear not only the forces of gravity, but also the side pressure of the wind. Mr. Young had stated that it was a simple matter to provide against the force of wind; but that was really the most difficult and complicated part of the problem. The most economical depth possible had to be used to resist the force of gravity; but then the side pressure prevented the use of an economical height; consequently the bridge, when it was finished, was a compromise between the results of the two forces. That was why the long-span bridges were comparatively not so high as those of shorter span. In spans of less than 200 feet the proportion was $\frac{1}{5}$ or $\frac{1}{6}$. The Kentucky bridge alluded to by Mr. Fox was really a discontinuous girder, and the proof of it was, that, when the test loads ran over it, there was a movement of the lower chords at the point where they were cut. The comparing of the moduli of elasticity of the irons was simply an experiment, and it was done in this case to test the very theory of discontinuous girders. With regard to the platforms, he was sorry to say that there were many bridges in America of which the platforms were not so strong as they ought to be; but every day approximation was made to the English practice in that respect. American engineers looked with the greatest respect and favour upon English practice; they admired the safety which English engineers gave to their bridges; and wherever it was possible, whenever bridges were planned by engineers, the platforms were now made much stronger than the girders; while the girders, in the long span, carried about 1 ton to the foot-load, the platforms, with the same strain, were made to carry from perhaps 2 to $2\frac{1}{2}$ tons. Mr. Matheson had spoken of the difficulty of rolling one of the bridges out. He could not think that that was an objection, when it was seen that it could be corbelled out. With regard to Mr. Phipps's objection to vertical members, the reason why American engineers preferred them was that there might be no reversal of strain. It would be easy to use triangular or Warren girders in which the members should be placed alternately in tension and compression—that they should be stiffened members and at the same

time made to resist tension; but in the bridges in question every member was strained only one way; the posts and the upper members were strained in compression; the lower members and the diagonals were strained in tension: there was no reversal of strains, and that was thought to conduce to the longevity of the bridge. Mr. Barlow considered that the modulus of elasticity of steel had been spoken of by him as uncertain. On the contrary, his views agreed entirely with those of Mr. Barlow, that the modulus of elasticity of steel was less variable than of iron. What was said by him was—"its strength is variable"; and that needed a little explanation. In forms of regular section, like plates and angles, the strength of steel was not necessarily variable; but where the sections were subject to sudden changes of form, as in eye-bars, the strength of steel was variable. In eye-bars of iron, as made at Phœnixville, and at several other large American bridge works, the eye was not welded to the bar, but the bar was upset, or dieforged under hydraulic pressure, into the exact shape required. Care had to be taken neither to burn the iron nor to distort its fibres. That was so well understood that specifications now demanded that iron eye-bars should always break in the bar, never across the eye. It had taken some time to learn how to do that in iron. If bridges were to be built of steel, engineers would not be willing to give up what they considered to be the very decided advantage of eye-bars and pins. Before they could venture to use steel eye-bars, a series of very careful experiments would have to be made: first, to determine the proper mode of manufacture of steel eye-bars, and then to ascertain the proper proportion of areas of eye to bar, and of the curves uniting the two. That had not been done yet, for the reason given by him, that the present price of steel was too high in America, as compared with that of iron, to make its use economical in girder bridges. In reference to the table, p. 194, it was stated that the strains on the Kuilenburg bridge, No. 15, were 11,800 in compression and 14,560 in tension. Mr. Van Diesen, the engineer, had stated that those were the strains on the cross-girders, which were of Bessemer steel, and no such strains existed in the main girders. He did not say what the maximum strains were, but if they did not exceed 4 tons per square inch in compression and 5 tons in tension, then the comparison with the Ohio bridge, No. 16, would be more favourable. It was believed that equating the two bridges for lengths, widths, loads carried, and strains per square inch, would reduce the excess of weight of No. 15 over No. 16 at least one-half of the amount previously given, or, say, to 300 tons. Mr. Baker

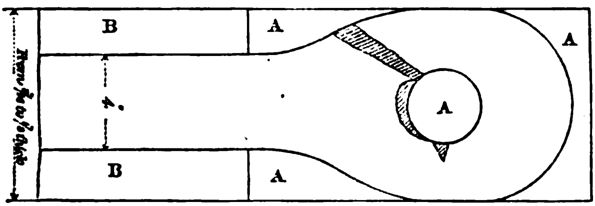
had pointed out that an error in length of an eye-bar of $\frac{1}{64}$ inch might increase the strain $1\frac{1}{2}$ ton per square inch. It might increase it much more than that. If a certain load was carried by a set of eye-bars in any frame, and one bar should be $\frac{1}{64}$ inch shorter than the others, it would have to carry the whole load belonging to the four unless it stretched $\frac{1}{64}$ inch. But that irregularity of lengths never had occurred, at least in his practice. Care was taken that all eye-bars and all struts intended to be of the same length should be exactly of the same length. The process of manufacture ensured that. The holes were drilled at each end of the eye-bar at the same time on a tool with a wrought-iron bed long enough to support the longest bar. The struts also were milled to length at both ends simultaneously. The limit of error of $\frac{1}{64}$ inch referred to was that allowed in transferring dimensions from the drawings to the template or machine tool. Eye-bars 45 to 55 feet long were made at Phœnixville, so that a dozen might be piled on the top of each other. A long pin exactly filling the holes was put through them at one end, and if another similar pin would not go smoothly and easily through all the holes at the other end, the bar which should refuse to admit the pin would be rejected. Such accuracy of workmanship cost no more than ill-fitting work when once the machine tools and proper appliances were prepared. The application of machinery to the construction of pin and eye-bar bridges was of the same kind as that now applied to the construction of rifles, sewing-machines, &c. His experience led him to believe that the fears sometimes expressed of loose joints were quite unfounded when bridges were made in that way.

Mr. Berkley described, through the Secretary, the way in which bars had been made for him by Messrs. Westwood, Baillie, and Co., since Messrs. Howard and Ravenhill had ceased to manufacture them.

From a bar of iron (Fig. 8), the full width of the swelled head, the portions A A A were stamped out at the same time under the steam hammer. The pin-holes, stamped at the same operation, were about $\frac{3}{4}$ inch less in diameter than the finished size, being afterwards bored out to the requisite dimensions. After the heads had been thus formed, the links were taken to a planing machine, and the two strips B B were planed off the bar in the usual manner. The pin-holes being then bored out to the finished size at one end, a number of the links were stacked on each other, the links being dropped over a stud of the exact size of the pin; and

the pin-holes at the opposite end were then bored out. The shape of the heads of the links was in accordance with the proportions stated in his Paper on "The Strength of Iron and Steel."[1] Some links had been tested; three of them broke at the head, as shown in Fig. 8, five through the bars, and ten did not break, but

Fig. 8.



stretched more than 9 inches in the length of the bar, which was 8 feet from centre to centre of the pin-holes, being the limit of the machine. The maximum and minimum breaking strains were 21·8 tons and 20·9 tons per square inch of section of bar; but one of the bars which did not break bore a strain per square inch of section of bar equal to 22·22 tons.

Mr. W. H. Bidder observed, through the Secretary, that the Author appeared to have rather over-estimated the quantity of material required to span an opening of 720-feet clear, to carry a double line of way. In his first example of an ordinary girder, he estimated the gross amount of iron required to be 4,500 tons; but he thought that this was in excess at least 500 tons, and that much of this excess was due to the great obliquity of the diagonals in the sides making them longer, and the strains on them greater than necessary. He was at a loss to understand why the diagonals were so much inclined towards the horizon, and also why the Author had adopted four main girders for his example. Mr. Bidder had found that, where the main girders, transoms, longitudinals, and deck-plating of a bridge were all of iron, for all practical purposes the quantity in tons of iron required might be estimated as follows:—For spans of from 100 feet to 300 feet, with either two or three main girders. Where there were only two main girders, the weight of each would be found by squaring the span in feet and dividing the result by 300, and the weight of the

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxx., p. 215.

transoms, longitudinals, and deck-plating, would be found by multiplying the span in feet by 0·6. For three main girders, the same formula for each main girder, in the case of two girders only being adopted, would apply to the centre one; and for each side girder the weight might be found by squaring the span in feet and dividing the result by 500. The weight of the transoms, longitudinals, and deck-plating, would be found by multiplying the span in feet by 0·44. If the weights of the main girders proved heavier than by calculation from the above formula, then there was something wrong in the design. But for spans above 300 feet the divisions might be somewhat reduced; and, in the case of the span of 720 feet, the divisors might be 340 and 520 respectively, instead of 300 and 200, as taken for spans up to 300 feet. The several cases would then stand thus:—

For two main girders only—

	Tons.
One main girder $\frac{\text{span}^2}{340} = \frac{720^2}{340} =$	1,620
One " " " " = ...	1,620
Transoms, deck, &c., span × 0·6 = 720 × 0·6 = . .	432
Add for a top and cross bracing, say	328
Total	4,000

For three main girders—

	Tons.
One main centre girder $\frac{\text{span}^2}{340} = \frac{720^2}{340} =$	1,620
One main side " $\frac{\text{span}^2}{520} = \frac{720^2}{520} =$	990
One " " " " " = ...	990
Transoms, deck, &c. = span × 0·44 = 720 × 0·44 = .	316
Add for top and cross bracing, say	304
Total	4,220

And in the case of four main girders—

	Tons.
One main girder = $\frac{\text{span}^2}{520} = \frac{720^2}{520} =$	990
Three " " " " = ...	2,970
Transoms, deck, &c. = span × 0·4 = 720 × 0·4 . .	288
Add for top and cross bracing, say	292
Total	4,540

The preceding calculations were based on the assumption that no portion of the metal was strained beyond 4 tons to the square inch in compression, and 5 tons per square inch in tension. But the top and bottom members, for several feet from the ends of the girders, could not be diminished to anything like the extent required to produce a strain of 4 and 5 tons per square inch respectively, as in the other portions of the girder; and, indeed, nearly all the compressive members were subjected to a less strain than 4 tons to the inch, in order to keep them from buckling, particularly when they were so long that they came under the laws of long columns. Thus the waste of material would increase with the greater number of girders; hence, for a span of 720 feet, when only two main girders were adopted, the gross weight was only 4,000 tons; with three main girders, it was 4,220 tons; and with four main girders, 4,540 tons, or just the quantity estimated by the Author; consequently the lesser number of girders ought to be adopted, particularly as, on account of the greater masses of material in the various members with fewer girders, there would not be so many surfaces subjected to the weather, and greater durability would be ensured.

Mr. C. Elwin observed, through the Secretary, that in bridges of long span any saving of dead weight was obviously of great importance. In flanges composed of plates riveted together, the joint covers formed a large item in this weight, the width of flange necessary to give stiffness, and to make up the large amount of sectional area required, was such, that the limit of length of the plates, when they were made the whole width of the flange, was soon reached, and the number of joint covers was proportionately great. But if each layer of plates was made up of two or more widths, one say 4 inches more and one 4 inches less than the half width of the flange; and if these wide and narrow plates were placed alternately on each side of the flange, so as to break joint, much longer plates could be used, and a corresponding reduction gained in the number of covers required without loss of strength, and with only a small increase in the amount of work to be done. He had adopted this system in the main girder flanges of the floating dock recently constructed for the Victoria Graving Docks, the width of flange being 72 inches, and each layer containing one plate 40 inches and one plate 32 inches wide. He was not aware whether this system had been used before in the flange of a girder.

Sir C. A. Hartley remarked, through the Secretary, that having inspected some of the most important railway bridges in the United States, he was able to endorse the Author's statement that iron railway bridges of long spans in that country were conspicuous for economy of design, and generally for excellence of workmanship. In the Session of 1874–5 the Council of the Institution did him the honour to publish as a Selected Paper, his "Notes on Public Works in the United States and in Canada."[1] In that Paper short accounts were given of Mr. Albert Fink's railway bridge across the river Ohio, at Louisville (not at Cincinnati, as written by mistake in the "Notes"), and of Mr. C. Shaler Smith's railway bridge across the Missouri at St. Charles. He now desired to refer to these two bridges as being notable types of American iron bridges of large spans, the Louisville bridge having amongst its twenty-seven spans one span of 400 feet and one of 370 feet; and the St. Charles bridge having four spans of 320 feet and three of 305 feet. In the Louisville bridge the total weight of wrought and cast iron in the 400-feet span was 627 tons, and the total dead weight of the span, including its two over-hanging side-paths, was 743 tons, or 4162 lbs. per lineal foot. The bridge, in addition to its own weight, was proportioned for a rolling load of 2,600 lbs. per lineal foot, and with this maximum load the factor of safety in the cast-iron chords was from 6 to 7, and in the wrought-iron braces from 5 to 6, by Hodgkinson's formula. The depth of truss of the 400-feet span, which Colonel Fink called "a modified triangular truss," was 46 feet, or less than $\frac{1}{8}$ of the span; the proportion in the wrought-iron trusses of the Cincinnati and Kentucky bridges, specially cited by the Author, being $\frac{1}{10}$ of the span. The Paper was a most valuable one, and would doubtlessly be read with special interest by railway engineers in India, where as yet only six bridges had been erected, as far as he could learn, whose spans exceeded 180 feet, *i.e.*, Mr. Bradford Leslie's bridge across the Gorai,[2] which, besides smaller openings, had seven spans of 185 feet each; the Faun bridge, on the Great Indian Peninsula railway, which had four 196-feet openings in addition to two smaller ones; the bridge across the Chumbul at Dholpore, which had twelve spans of 200 feet and two of 150 feet; the bridges across the Jumna at Allahabad and Delhi, the spans of

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xl., p. 163.
[2] *Vide Ibid.*, vol. xxxiv., p. 1.

which were 220 feet; and the bridge across the Sutlej at Bahawul pore, which had sixteen spans of 264 feet each. With the exception of the Bahawulpore bridge, the foundations of which were carried down to a depth of 100 feet, no bridge in the Punjab had greater spans than 142 feet, the precise length, reckoning from centre to centre of the piers, of each of the sixty-four wrought-iron trussed spans of the famous bridge across the river Chenab at Wuzerabad. The spans of the bridges across the Jumna at Saharanpore, the Beas at Beas, and the Sutlej at Phillour, did not exceed 112 feet each; whilst the spans at Lahore across the Ravi, and at Jhelum across the Jhelum, did not exceed 100 feet.[1] After a close inspection of the railway bridges between the Jumna and the Jhelum, in the spring of 1877, whilst fully recognising the admirable manner in which the work had been constructed, he could not help carrying away with him the conviction that if much bolder spans had been adopted more satisfactory results would have been obtained at a far less ultimate cost. In his opinion, the want of sufficient waterway and the general adoption of short spans were the principal causes of the failures that took place nearly every rainy season in the foundations in one or more of the bridges spanning the great rivers flowing through the alluvial plains of India, notwithstanding that the brick cylinders on which the superstructures rested were carried down to depths often exceeding 70 feet below the level of low water. In some cases ¼ of the waterway was taken up by the too thickly planted piers; and in other cases the evil, inherent to the presence of so many solid obstructions to the free play of the current, was greatly aggravated by the necessity which often presented itself during floods of throwing down masses of rubble-stone, or "kunkur," round the piers to guard their foundations against undue scour, which had been known to scoop out holes 30 feet deep in the space of only a few hours. The ultimate consequences of such additional encroachments on the waterway, if carried out on a large scale, were not difficult to foretell. The lesson taught by the examples referred to seemed to be that in dealing with the bridging of rivers whose beds to an unknown depth had been formed by their own deposits, and where, as in the Punjab, the channels were constantly changing in direction and in depth, owing to the violence of sudden floods and the absence of well-defined

[1] *Vide ante* pp. 61 and 94.

river banks, the adoption of wide spans instead of narrow ones would ensure greater safety to the structure itself, greater economy in its maintenance, more safety and freedom to the navigation, and, lastly, greater economy in the first cost and future maintenance of the bunds and spurs required in any case to keep such a supposed river, near the bridge, in proper train.

Mr. C. Shaler Smith stated, through the Secretary, that the ice freshet and flood, which destroyed the works at the Ohio bridge, occurred also at the Kentucky river, but owing to the difference in the systems of erection the latter bridge was pushed forward in defiance of the weather, and was completed two months later. The recorded deflections in the first and second conditions of the testing of the Ohio bridge, were, in his opinion, more than doubtful. The observations were taken in a driving rain, and the rodman placed on this span was very nervous on account of the great height. While they stood as the records of the official tests, neither Mr. Bouscaren, Chief Engineer of the Cincinnati Southern railroad, nor himself considered them as correct. On the following day a train of nearly the same weight only deflected the middle span $1\frac{3}{4}$ inch, rendering it almost certain that the two observations of the previous day were erroneous. During three months of the time occupied in the erection of the Ohio bridge, the weather was so exceptionally severe that the work on all the other bridges along the line was suspended. The most signal example of economy shown by the Table of particulars of various bridges (pp. 18 and 19) was that over the Susquehanna river. The Steubenville span was proportioned for 4,000 lbs. working load per foot. Bearing this in mind, it was not so far behind the others as it appeared to be at the first glance. The longitudinal stability of the Kentucky iron piers was required to be such that a train weighing 1,125 tons, with a brake applied to every wheel, and the wheels all sliding on the rails, should not produce tension at any part of the pier leg or base. The specifications in the Appendix to the Paper required that the piers should have much larger bases to meet lateral and longitudinal strains than had been given to the piers of the great French and Swiss viaducts, in any of which serious tension would be produced during such storms as were common in America. In the case of the Kentucky river bridge, it was provided that there should be added to the strains produced by the rolling road 30 per cent. for rail joists, 25 per cent. for floor beams, and 20 per cent. for counter rods. Also a pressure of wind of $31\frac{1}{2}$ lbs. per square foot on twice the surface of the

spans and iron piers, plus 315 lbs. per lineal foot on the train extended entirely across the bridge. When exposed to these lateral forces the piers were so proportioned as to be without tension at any part of the base.

Since the Paper was read information relating to other iron bridges of large span has been supplied to the Secretary, from which the following Table has been compiled.

SUPPLEMENTARY TABLE of GIRDER BRIDGES,

Spans exceeding

Number.	Date of Erection.	Where Built.	Name of Engineer.	Clear Span in Feet between Points of Bearings.	Tons of Iron (2,240 lbs.)	Dimensions in Feet. Width between Centres of Girders.	Panels. Number.	Panels. Length.	Panels. Height.
						Ft. Ins.		Ft. Ins.	Feet. Ins.
1	1870	Zeglin river, Stettin . .	Schröder	302	449	27 3	17	19 8	19 8 / 45 11
2	1869	Mersey river, Runcorn. .	W. Baker	305	702	28 0	Lattice.		27 0
3	1872	Rhine river, Rheinhausen	Hartwich	317	..	..	..	..	..
4	1876	Memel river, Tilsit . .	Schwedler	317	604	28 10	18	17 7	17 0 / 39 1
5	1873	Vistula river, Thorn . .	Schwedler	319	605	37 9	18	13 6 / 18 3	24 7 / 46 2
6	1869	Elb river, Hamburg .	Lohse	325	602	27 6	26	12 4	10 3
7	1874	Elb river, Totschen .	Gerlich	328	420	16 5	20	16 5	32 10
8	1872	Rhine river, Wesel . .	Dreling	334	..	27 5	28	12 4	18 5 / 37 0
9	1875	Elb river, Hohnsdorf .	Grüttefien	338	592	27 3	20	17 8	23 7 / 49 3
10	1870	Maas river, Crèvecœur .	..	341	512	16 8	23	14 7	23 0 / 41 0
11	1864	Old Rhine river Griethausen.	Hartwich	342	493	15 0	40	8 4	25 3
12	1870	Theiss river, Algyo . .	Körösi	342	452	16 5	20	17 0	34 2
13	1870	Rhine river, Mayence (New)	Pauli	345	352	15 1	13	26 3	0 0 / 49 3
14	1870	Rhine river, Düsseldorf .	Pichier	347	659	28 0	27	12 4	22 7 / 44 5
15	1878	Rhine river, Coblence (New)	Hilf	348	..	..	..	..	..
16	1871	Hollandsch-diep river, Moerdyk. .	A Commission	349	447	16 5	25	14 6	19 8 / 39 8
17	1869	Waal river, Bommel. .	G. van Diesen	408	860	17 2	27	14 11	23 0 / 42 7

CONSTRUCTED of IRON, for RAILWAYS.

300 feet.

Loads in lbs. per Lineal Foot.		Strains in lbs. per Square inch of Area.				Test Load.	Centre Deflection.	Dead Load of Iron, Timber, &c.	Deflection of Span from its own Weight on removal of Scaffolding.	Remarks.
		Tensile.		Compressive.						
Dead Load of Iron and Timber in Track.	Live Load of Engines and Cars.	From Constant Load only.	From Total Constant and Moving Loads.	From Constant Load only.	From Total Constant and Moving Loads.	Tons.	Inch.	Tons.	Inch.	
3,333	4,024	..	10,667	..	10,667	..	..	..	..	Double track.
5,667	3,360	6,618	10,416	4,883	7,795	341	1¼	..	..	All wrought iron— Estimated camber 9 in. Actual „ 9½ „
..	..	..	..	..	..	..	..	..	..	
4,700	5,097	..	10,667	..	10,667	..	..	..	..	Double track.
4,252	4,830	..	10,383	..	10,383	..	..	..	..	Double track.
4,144	4,830	..	10,667	..	10,667	..	..	..	..	Double track.
2,864	2,683	..	11,378	..	11,378	354	$1\frac{53}{100}$	..	..	Single track.
..	..	..	..	..	..	..	..	..	..	Double track.
4,292	4,830	..	..	..	..	..	..	..	..	Double track.
3,455	2,109	6,000	9,598	6,000	9,598	321	$1\frac{4}{25}$	527	..	
3,413	2,140	..	10,383	..	10,383	214	$1\frac{43}{100}$	..	..	Single track.
2,958	2,132	..	11,378	..	11,378	312	$1\frac{38}{100}$	..	..	Single track.
2,455	2,146	5,689	11,378	5,689	11,378	..	..	..	..	
,239	4,272	..	10,383	..	10,383	..	..	..	..	Double track.
..	..	..	..	..	..	..	..	..	..	
,212	2,222	5,042	8,533	5,042	8,533	336	1½	486	..	
,412	1,935	6,378	9,243	6,378	9,954	352	$\frac{8}{25}$	816	..	

May 28, 1878.

JOHN FREDERIC BATEMAN, F.R.SS. L. & E., President, in the Chair.

The following Candidates were balloted for and duly elected:—

Albert Duncan Austin, Thomas William Horn, William Heerlein Lindley, Henry Augustine FitzGerald MacLeod, Thomas Frederick Parkinson, Gerald Edward Smith, and Patrick Stirling, as Members; Walter Bentley, Rev. James Oliver Bevan, M.A., John George Blackett, John Edward Corry, Charles Fowler, William Griffiths, Charles Horsley, Willoughby Rochester Hughes, Stud. Inst. C.E., Charles Lavey, Edwin Lee, Henry Charles Litchfield, John Macnie, John Monthermer Montague, B.A., Stud. Inst. C.E., John Curwen Pottinger, Hans Roderich Leopold Reincke, Michelangelo Sclaverani, John Tunstall, John Warren, and Robert Henry Willis, Stud. Inst. C.E., as Associates.

It was announced that the Council, acting under the provisions of Sect. III., Cl. 8 of the Bye-Laws, had transferred Samuel Abbott, John Brunlees, Henry Davey, John Dickson Derry, Ben James Fisher, Charles Edward Gael, B.A., Walter Hunter, Ailsa Janson, Thomas Miller, Robert Andrew Robertson, and Alexander Smith, from the class of Associate to that of Member.

Also that, under the provisions of Sect. IV. of the Bye-Laws, the following Candidates, having been duly recommended, had been admitted as Students of the Institution:—John Tipping Gardner, James Bernard Hunter, Stewart Kershaw, and James Brown Stephen.

The discussion on the Paper, No. 1,574, "The Design generally of Iron Bridges of very large Span for Railway traffic," by Mr. T. C. Clarke, occupied the entire evening.

June 3, 1878.

This was the fiftieth anniversary of the grant of the Royal Charter of Incorporation, when the Session was concluded by a *conversazione*, which was given by the President at the India Museum, South Kensington, by permission of the Secretary of State for India.

Sect. II.—OTHER SELECTED PAPERS.

No. 1603.—"A Skeleton Pontoon Bridge." By Bagot William Blood, M. Inst. C.E.

During the spring and summer of 1868 a suspension bridge was being erected over the Jumna, on the hill road between Mussourie and Simla, in the Himalaya. The road was then under construction, and the only means of crossing the river at all seasons consisted of a rope carrying a basket, which was hauled at considerable elevation from side to side—an inconvenient and unpleasant mode of transit, and suitable simply for men and other light loads. There was considerable traffic over the completed portion of the road between Mussourie and the new Military Hill station of Chukrata, and cattle and horses had to be pushed into the water, and compelled to swim the rapid river. Heavy loads were sent by another road, about 24 miles longer.

A temporary trestle bridge was used during the season when the river was low, but this was invariably swept away in March, and could not be kept up without great expense until the middle of October. As, during the erection of the suspension bridge, men and heavy materials had to be carried across frequently, it became necessary to devise some better communication than was afforded by the rope jūla. The chief cause of destruction to the trestle bridge was the floating down the river of large logs of drift timber; in fact whole pine trees (the branches of which had been broken off in the rapids above) were carried against the light structure with great violence.

After several attempts to keep up the trestle bridge, and, by means of a floating protection above, to prevent the logs reaching it, the Author decided to construct a light pontoon bridge, and to moor it in such a way as to withstand the shocks of the drift timber. The only difficulty was to build the pontoons. It was necessary that they should be cheap, very strong, light, and made up from material then on the ground. The up-stream end required to be inclined to the surface, like the bows of a Thames punt, so as to pass the floating logs under. The only available material at all suited to the purpose was a quantity of deodar planking, 3 inches thick, cut in lengths of 10 feet, which was intended for

the flooring of the suspension bridge, and this could not be cut in any way which would prevent its being eventually so employed.

There was an exceedingly rapid reach of the river near the site of the suspension bridge; and it occurred to the Author to make use of the power developed by the water at a high velocity to support the temporary bridge. After calculation and experiments it was decided to use the pontoon bridge shown in Plate 10.

The pontoons were constructed by placing two planks, each 10 feet long and 3 feet by 3 inches in section, 15 feet apart, and framing them strongly together with ordinary round timber, which was cut in the forest near the bridge, each plank being held on edge at an angle of about 54° from the vertical, both inclining up stream. These planks formed the bow and stern of the pontoon. When the pontoons were moored in the stream, which had a velocity of from 10 to 14 feet per second, according to the state of flood, the pressure of the water upon the inclined surfaces of the planks enabled the pontoons to carry a considerable load. On a log coming in contact with the bridge, the pontoons struck lifted slightly and allowed the log to pass. The pontoons were moored to a strong chain hung across the river, 10 feet above the level of the highest flood. The object of hanging the mooring so high was to assist the pontoons in lifting when struck by a log.

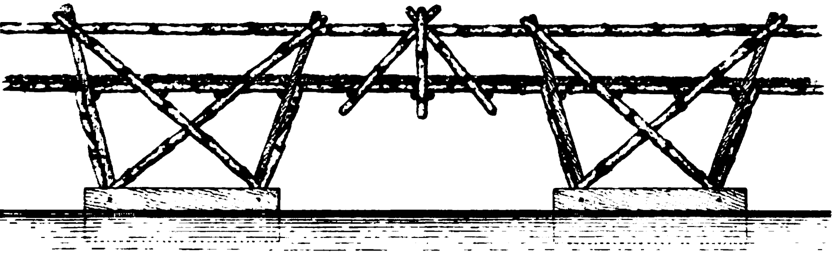
The pontoons were fixed at a distance apart of 25 feet from centre to centre, and the clear span between them was therefore 15 feet. Light trestles resting upon the pontoons carried the roadway at a height of 6 feet above the water. The roadway was constructed of strong bamboos, laid close together, over which was lashed transversely split bamboo, and the whole covered with brushwood and grass as a kind of mat. A hand-rail was made on each side by lashing strong bamboos horizontally to uprights from the pontoons, and attaching them to the roadway between the trestles, thereby adding slightly to the strength of the road. This arrangement gave a good flexible way, which allowed of considerable independent vertical movement of the pontoons. The bridge was of such a length that some of the pontoons lay above water on the bank, and were only floated by high floods.

This bridge was very successful, and took the whole traffic of the road, as well as that caused by the erection of the suspension bridge, until the completion of the permanent work.

The communication is accompanied by three diagrams, from which Plate 10 has been compiled.

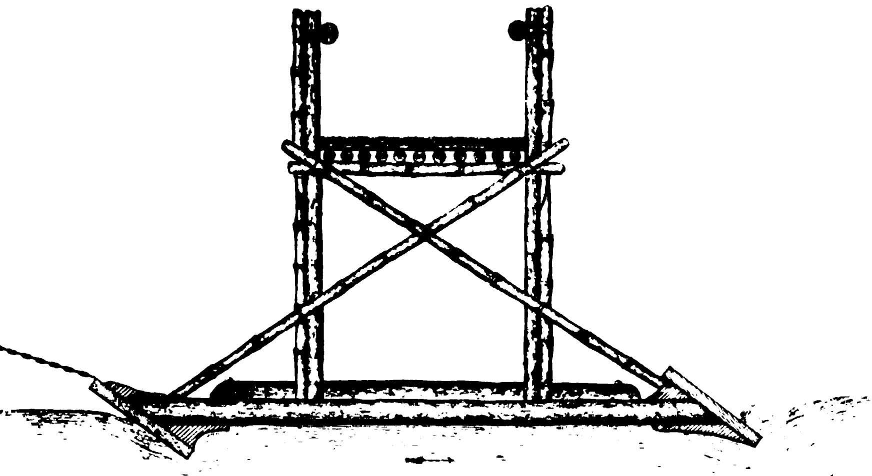
A SKELETON PONTOON BRIDGE.

ELEVATION OF ONE BAY.

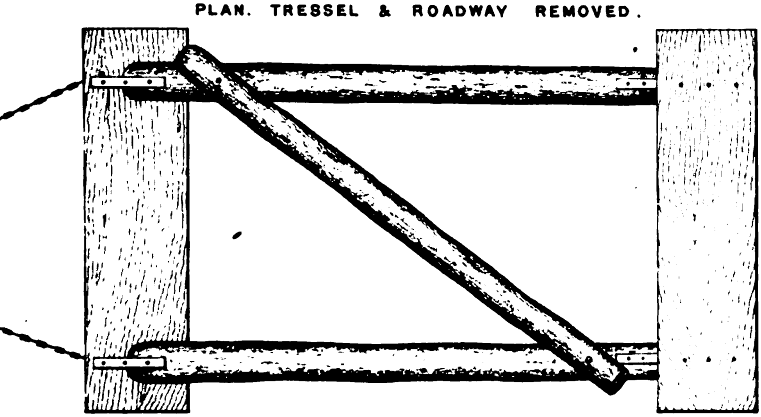


Scale 10 ft to the Inch.

END VIEW OF PONTOON.



PLAN. TRESSEL & ROADWAY REMOVED.



Scale 5 ft to the Inch.

..OOD.

No. 1,562.—"Portland Cement Concrete." By John Watt Sandeman, M. Inst. C.E.

In this Paper the relative values of concretes, according to the volumes of the materials used in their composition, are deduced from experiments, the results being given in a tabular form.

In regard to the value of concrete, it is proposed that it should be compared upon the basis of relative strength, as, with the same nature and dimensions of broken materials, the relative strength of concrete depends upon that of the mortars which unite the aggregates. From Table 1 it will be observed that two or more concretes can be made with the same relative volume of cement to each, but if one be made with a proportionately larger volume of sand, and lesser volume of aggregates, it can be produced in most instances at less cost, dependent upon the relative cost of the sand and aggregates; at the same time its relative strength will be less, and therefore under most conditions its volume would have to be increased to afford the same amount of cohesive and compressive strength, the basis of comparison (Table 1, column 6) being taken from Mr. Grant's Tables of the strength of cement compos.[1]

To produce concrete of maximum strength with a given ratio of cement to the other materials, it is evident that the less the volume of mortar used the less will be the quantity of sand required, and the greater the proportionate strength of the mortar. It is therefore of importance to determine the minimum volume of mortar necessary to fill the interstices of the aggregates. This is deduced from the results of the experiments given in Table 2, which show that the volume of the interstices (and consequently of the mortar) bears a regular ratio to the volume of the aggregates, according to the dimensions and form of the latter; and that a less volume of mortar than that which has been deduced from the experiments would not fill the interstices of the aggregates, and consequently would not produce watertight concrete.

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxv., p. 66, and vol. xxxii., p. 266.

TABLE 1.—COMPARISON of the VALUE of PORTLAND CEMENT CONCRETES MADE UPON the BASIS of THEIR RELATIVE STRENGTH.

1	2			3	4	5	6					7	8
Number of Example.	Relative Volumes of the Materials composing the Concretes.			Volume of the Mortar (when set) in percentage of the Volume of the Concrete.	Proportion of Cement to the other Materials composing the Concretes.	Actual Cost[1] of the Concretes per Cubic Yard.	Relative Strength[2] of Cement Compos or Mortars taken from Mr. Grant's Tables of Experiments.					Estimated Value of the Concretes, in Proportion to the Relative Strength of the Mortars. The Cost of Nos. 1, 3, and 5 Concretes being the Units of Comparison; the Proportions of these having been deduced from the Experiments (Table 2). Per cubic yard.	Estimated Loss in Value, occasioned by providing an unnecessarily large Volume of Mortar in Nos. 2, 4, and 6 Concretes. Per cubic yard.
							Cohesive Strength[3] from Table of Experiments No. 17, Vol. xxv., p. 88.		Compressive Strength[4] from Table of Experiments, No. 4, series C, Vol. xxxii., p. 287.		Basis of Comparison or Average Relative Strength of the Mortars. Per cent.		
	Cement.	Sand.	Aggregates.				Strength in lbs. per 2¼ square inches.	Relative Strength. Per cent.	Strength in Tons per Brick.	Relative Strength. Per cent.			
						£. s. d.						£. s. d.	£. s. d.
I.	1	1	3·16	42·40	1 to 4·16	1 6 0	700·3	100·00	63·26	100·00	100·00	1 6 0	..
II.	1	2	2·16	61·73	1 „ 4·16	1 4 1	458·5	65·47	41·48	65·57	65·52	0 17 0	0 7 1
III.	1	2	4·74	42·40	1 „ 6·74	1 0 2	458·5	65·47	41·48	65·57	65·52	1 0 2	..
IV.	1	3	3·74	55·40	1 „ 6·74	0 18 11	320·6	45·78	26·50	41·89	43·83	0 13 6	0 5 5
V.	1	3	6·32	42·40	1 „ 9·32	0 17 0	320·6	45·78	26·50	41·89	43·83	0 17 0	..
VI.	1	4	5·32	52·19	1 „ 9·32	0 16 4	221·6	31·64	19·82	31·33	31·48	0 12 2	0 4 2

[1] Cost at the Weaver Navigation. (See Table 6.)

[2] The strength of concretes would of course vary with different natures and dimensions of aggregates; also the strength of mortars, when the cement is mixed with different natures and sizes of sand, as shown by Mr. Grant's experiments.

[3] The cohesive strength of compos, made from cement and sea sand is that which has been selected from the Table.

[4] The compressive strength of compos, given in the second last column of the Table referred to, is that which has been selected.

It is also evident, if the mortar were increased in volume beyond what is required to fill the interstices, and to separate all the stones, by adding to the sand and proportionately diminishing the aggregates, that while the volume of the cement would bear the same proportion to the volume of the concrete, yet a weaker mortar, and consequently a weaker concrete, would be produced without a proportionate reduction in the cost of the latter. Thus the strongest concretes (compared relatively) are the most economical. No. 1 concrete is much stronger in proportion to its actual cost than any other.

It may be contended that concretes which can be produced at somewhat less actual cost, by using more mortar without increasing the relative volume of cement, although weaker, would still possess sufficient constructive strength for certain works, and that it would be more economical to employ them. This may be true for exceptional work; but it does not lessen the importance of the principle of comparison sought to be established, which will apply to all concretes used for works in which a definite amount of either cohesive or compressive strength is a *sine quâ non*. If for such works concretes possessing an unnecessarily large amount of mortar were used, the estimated loss in their value would be even greater than is represented in Table 1, as additional excavation and other expense would be involved in providing for the larger quantity of concrete which would be required.

It has been assumed that if the whole of the interstices of the aggregates be filled with mortar, watertight concrete will be produced; but this is on the assumption that the mortar when set is watertight. To ensure this being the case it is requisite that the cement when gauged be in sufficient quantity to fill the interstices of the sand.

The interstices of silicious sea-sand, when not compressed, were ascertained by the Author to amount to about 40 per cent. of the volume of the sand. For fine or coarse sand, or a mixture of the two, the volume of the interstices did not vary much. When the sand was compressed with a rammer in water, its volume could be reduced to the extent of 12½ per cent. If, therefore, a volume of cement be taken equal to the volume of the interstices of the uncompressed sand, it will possess a sufficient surplus to ensure of there being a film of cement around each particle of sand. Therefore to produce concrete impermeable to water, from Portland cement, coarse or fine silicious sand, and

broken red sandstone (of the dimensions given in Experiment No. VII., Table 8), the relative volumes of the two latter materials to 1 of cement should not exceed $2\frac{1}{2}$ of sand and $5\frac{1}{2}$ of broken red sandstone, which proportions agree with those given in Table 3.

Impermeability to water will also to a certain extent depend upon the thickness of the concrete through which the water might have to pass, as well as upon the pressure of water; as a depth of water of a few feet was found to percolate even through the solid red sandstone. This circumstance renders red sandstone an excellent filter, for which it is frequently used in Cheshire.

For reservoir walls, dry docks in water-bearing strata, &c., concretes of the foregoing proportions would be requisite; but for works in which watertightness is not essential, weaker concretes containing larger volumes of sand and aggregates are suitable, as in the other proportions given in Tables 3 and 10.

The concrete referred to in the following description is only cited in explanation of the mode of proportioning the materials. The character of the broken stone is similar to that adopted with success for concrete on a large scale by Mr. G. F. Lyster, M.Inst. C.E., at the Birkenhead docks. The stones appeared to be of about the largest dimensions which it would be advisable to mix into concrete by machinery. The relative proportions of cement and sand are only applicable to one class of aggregates. For other descriptions of aggregates, while the same system of measurement might still be adopted, other proportions of cement and sand would be required. These proportions are given in Table 10, having been deduced from the results of the experiments with various aggregates given in Table 8.

When concrete is in large masses, and it is desired to introduce rubble blocks besides the smaller aggregates, an additional quantity of mortar should be provided, proportional to the number of blocks introduced. This quantity can readily be calculated from the number and average dimensions of the blocks, allowing sufficient mortar to make a thick joint around each. The number of blocks capable of being introduced into any concrete will have to be determined by actual trial in each case, as they will be limited according to the dimensions of the smaller aggregates contained in the concrete.

If it be desired to use Thames ballast, and at the same time to produce watertight concrete, the proportion of sand to gravel may

be ascertained by screening a few samples and taking the average. If the sand should be in excess of the proportions stated, it can be calculated whether it would be more profitable to screen the ballast and dispense with some sand, or to increase the volume of the cement.

In submitting the results of the following experiments, and the description of a mode of proportioning the materials for concretes which have been deduced from them, the Author has only aimed at recording data sufficiently accurate for practical purposes. There is no reason to suppose that the results would have been much modified, had the experiments been conducted upon a larger scale.

The mode of proportioning the volumes of the materials may be described thus :—

First.—To ascertain the proportion which should subsist between the mortar and the broken stone.

The insterstices between pieces of broken red sandstone, varying in size up to what would pass through an 8-inch ring, are found to be 36 per cent. of the whole volume (see Experiment No. VII., Table 8). But as the stones were in contact with each other when this measurement was made, a percentage has been deducted from their volume and added to that of the interstices, to obtain the volume of mortar which would be sufficient to separate the stones at all points, and to ensure the complete filling of the whole of the intersticial space. For ordinary concrete this may be taken at 10 per cent. of the volume of the stone.[1] Dividing this additional mortar throughout 90 per cent. of a volume of broken stone, it would afford about $\frac{1}{8}$ inch as the minimum thickness of a mortar-joint at any point, assuming the average diameter of the stones to be $3\frac{1}{2}$ inches. The measure of the interstices of 90 per cent. of a volume of broken stone (the proportion for one volume being 36 per cent.) is 32·4 per cent., and adding the 10 per cent. allowed for the separation of the stones, gives 42·4 per cent. as the proportion which the mortar should bear to the 90 per cent. of broken stone contained in one volume of concrete.

Second.—To ascertain the volumes of the dry materials required for the mortar.

The proportion previously ascertained is the volume of the

[1] For concrete to be placed under water the proportion should be increased to 15 per cent.

mortar when set hard, but the dry materials of which it is composed contract considerably in volume.

TABLE 2.—CONTRACTION OF DRY MATERIALS WHEN MADE INTO MORTAR.

—	Proportions.		
	1 Cement to 1 Sand.	1 Cement to 2 Sand.	1 Cement to 3 Sand.
	Per cent.	Per cent.	Per cent.
First.—By admixture with water . . .	15·00	16·66	17·50
NOTE.—These percentages are the averages of 10 per cent. for the contraction of cement, and 20 per cent. for the contraction of sand (see experiments Nos. 1 and 2, Table 9).			
Second.—By admixture with each other .	5·00	5·00	5·00
(See note appended to Table 9.)			
Third.—By the cement in setting to hardness, from the condition of mortar . .	4·00	4·00	4·00
NOTE.—The contraction by setting of cement mortar, when made of different proportions of cement and sand, was found to be nearly the same as for neat cement.			
Total ratios of contraction of the materials, in percentage of their own volume . .	24·00	25·66	26·50
Total ratios of contraction of the materials, in percentage of the volume of the mortar when set.	31·58	34·53	36·05
Total volumes of the dry materials (cement and sand), in percentage of the volume of the concrete, necessary to produce the 42·4 per cent. of mortar	55·79	57·04	57·68

From the foregoing it appears that the total percentages of dry cement and sand required for different proportions of mortar vary so little that it will be sufficient to take the average, viz., about 57 per cent., an amount equal to 14·6 per cent. more than the volume of the mortar when set required to 1 volume of concrete.

Third.—To ascertain the relative volumes of the dry materials, which should compose concretes of different proportions.

It has been shown that 57 per cent. of dry cement and sand are required to produce mortar sufficient for 90 per cent. of broken stone contained in 1 volume of concrete; or 1 volume of the first

to 1·58 volume of the second; this will give concretes of the following proportions. (See also Table 10.)

TABLE 3.—RELATIVE VOLUMES of DRY MATERIALS in the COMPOSITION of CONCRETES of DIFFERENT PROPORTIONS.

——	Cement Volume.	Sand Volumes.	Broken Stone Volumes.
1 volume cement to 4·16 volumes sand and stone	1	1·0	3·16
1 " " 5·45 " " "	1	1·5	3·95
1 " " 6·74 " " "	1	2·0	4·74
1 " " 8·03 " " "	1	2·5	5·53
1 " " 9·32 " " "	1	3·0	6·32
1 " " 10·61 " " "	1	3·5	7·11

Fourth.—To ascertain the weight and cost of the dry materials contained in 1 cubic yard of concrete.

As before 1 volume of concrete should contain 57 per cent. of its volume of dry cement and sand, and 90 per cent. of its volume of broken stone, or 57·6 per cent. of cube-stone; these together amount to 14·6 per cent. more than the 1 volume, due to the additional quantity of dry cement and sand necessary to produce the 42·4 per cent. of mortar. Taking the 1 volume of concrete as a cubic yard, the 57 per cent. would be equal to 15·39 cubic feet, to be divided in proportion to the relative volumes of the cement and sand; and in the case of concrete made in the proportion of 1 to 6·74 it would be 1 of cement to 2 of sand, or 5·13 cubic feet of the former to 10·26 cubic feet of the latter. The 90 per cent. of broken stone would be equal to 24·3 cubic feet. The weights of the dry materials contained in 1 cubic yard of concrete would therefore be as follow:—

TABLE 4.—CONCRETE MADE in the PROPORTION of 1 to 6·74, or 1 of CEMENT to 2 of SAND, and 4·74 of BROKEN STONE.

——	Weights of the Dry Materials contained in 1 cubic yard of Concrete.	Proportionate Weight of the Materials, taking Cement as the unit of comparison.
	lbs.	
5·13 cubic feet of cement × 94 lbs.	482·22	1·00
10·26 " " " sand × 80 "	820·80	1·70
24·30 " " " broken stone × 91¼ "	2217·37	4·60
Total weight of the materials per cubic yard	3520·39 or 1·57 ton.	7·30

TABLE 5.—CONCRETE MADE in the PROPORTION of 1 to 9·32, or 1 of CEMENT to 3 of SAND, and 6·32 of BROKEN STONE.

—	Weights of the Dry Materials contained in 1 cubic yard of Concrete.	Proportionate Weight of the Materials taking Cement as the unit of comparison.
	lbs.	lbs.
3·85 cubic feet of cement × 94 lbs.	361·90	1·00
11·54 „ „ „ sand × 80 „	923·20	2·55
24·30 „ „ „ broken stone × 91¼ „	2217·37	6·14
Total weight of the materials per cubic yard.	3501·47 or 1·56 ton.	9·69

TABLE 6.—COST of PORTLAND CEMENT CONCRETE (at the WEAVER NAVIGATION) MADE in the PROPORTION of 1 to 6·74, or 1 of CEMENT to 2 of SAND, and 4·74 of BROKEN STONE. (See also Table 10, Concrete No. VII.)

—	Proportionate Weights of Materials as previously ascertained.	Rate.	—
	Tons.	£. s. d.	£. s. d.
Cement delivered at site of works . .	1·0	2 16 6	2 16 6
Sand „ „ „ . .	1·7	0 0 9	0 1 3
Broken red sandstone „ „ . .	4·6	0 4 9	1 1 10
Cost of . .	7·3	..	3 19 7
Cost of materials per ton			0 10 11
Cost of materials per cubic yard at 1·57 ton per cubic yard . .			0 17 2
Mixing 1 cubic yard of materials by Messent's mixer, including wheeling to mixer, pumping water, and wheeling to, and depositing *in situ*, when the total distance does not exceed 100 yards on the average			0 3 0
Cost of concrete per cubic yard			1 0 2

TABLE 7.—COST of CONCRETE MADE in the PROPORTION of 1 to 9·32, or 1 of CEMENT to 3 of SAND, and 6·32 of BROKEN STONE. (See also Table 10, Concrete No. VII.)

—	Proportionate Weights of Materials as previously ascertained.	Rate.			—		
	Tons.	£.	s.	d.	£.	s.	d.
Cement delivered at site of works . .	1·00	2	16	6	2	16	6
Sand " " " . .	2·55	0	0	9	0	1	11
Broken red sandstone " " . .	6·14	0	4	9	1	9	2
Cost of . .	9·69	..			4	7	7
Cost of materials per ton					0	9	0
Cost of materials per cubic yard at 1·56 ton per cubic yard . .					0	14	0
Mixing 1 cubic yard as before					0	3	0
Cost of concrete per cubic yard					0	17	0

In the experiments, the results of which are given in the following Tables, the materials were all measured in a cylindrical iron box having a flat bottom, measuring 1 foot deep inside, and 1 foot 7¼ inches in internal diameter, which thus contained exactly 2 cubic feet. The weight of the box was 56 lbs. The materials were filled into the box with a spade, and were not compressed in any way. Where any doubt existed as to the accuracy of the weight or measure of any materials, the experiment was repeated.

TABLE 8.—RESULTS of EXPERIMENTS MADE to ASCERTAIN the RELATIVE VOLUME of the INTERSTICIAL SPACES APPERTAINING to DIFFERENT KINDS and DIMENSIONS of BROKEN MATERIALS or AGGREGATES.

Number of Experiment.	Natures and Dimensions of Broken Materials or Aggregates.	Weight of Material per Cubic Foot.	Weight of Fresh Water necessary to fill Interstices of 1 Cubic Foot of Material.	Volume of Water necessary to fill Interstices of 1 Cubic Foot of Material.	Volume of Water or Volume of Interstices in Percentage of the Volume of the Material and its Interstices.
		lbs.	lbs.	Cubic foot.	Per cent.
I.	Welsh limestone broken by Blake's stone-crusher into irregular flat oblong pieces, most of which when gauged the narrowest way would pass through a 3-inch ring	95	31·75	0·509	50·9
II.	Gravel obtained from a bed near Northwich; screened free from sand, varying in size from small pebbles to pieces gauged by a 2½-inch ring.	111¼	21·00	0·336	33·6
III.	Welsh limestone and gravel well mixed in equal proportions, of the same dimensions as in experiments I. and II. . . .	113¾	21·25	0·34	34·0
IV.	Anglesea limestone (mason's shivers), varying in size from small gravel to pieces gauged by a 4-inch ring	90½	30·00	0·48	48·0
V.	Runcorn red sandstone (large) broken by hand, varying in size from pieces gauged by a 4-inch ring to pieces gauged by an 8-inch ring	74	31·25	0·50	50·0
VI.	Runcorn red sandstone (small) broken by hand, varying in size from sand to pieces gauged by a 4-inch ring	92	21·25	0·34	34·0
VII.	Runcorn red sandstone (large and small) mixed in equal proportions, of the same dimensions as in experiments V. and VI. .	91¼	22·50	0·36	36·0

NOTE.—From Experiments I., II. and III. it appears that broken limestone when unmixed weighs 95 lbs. per cubic foot, and gravel 111¼ lbs.; average 103¼ lbs.; when mixed they weigh 113¾ lbs.; difference 10½ lbs., = 10 per cent. reduction in volume.

TABLE 9.—RESULTS of EXPERIMENTS made to ASCERTAIN the REDUCTION in VOLUME of CEMENT and SAND, by ADMIXTURE with WATER to the CONSISTENCY of MORTAR.

Number of Experiment.	Material.	Weight of a cubic foot of the Material when gauged with Water to the consistency of Mortar (not including the Water of Admixture).	Weight of the Water of Admixture.	Weight of a cubic foot of the Dry Material.	Difference between Weight of Material gauged with Water and Material Dry.	Ratio of Reduction in Volume of Material by Admixture with Water, in percentage of the Volume of the Dry Material.
		lbs.	lbs.	lbs.	lbs.	Per cent.
I.	Portland cement (Rugby) .	104	30	94	10	10·64
II.	Silicious sand, clean and small grained, dredged from the bed of the River Weaver	96	16·5	80	16	20·00
III.	Portland cement and sand (same as above), mixed in equal proportions . . .	104	22	87 (average of the above)	17	19·54

NOTE.—From the above it appears that cement when gauged alone contracts about 10 per cent., and sand 20 per cent.; average of the two = 15 per cent. When mixed they contract about 20 per cent.; difference = 5 per cent. Experiments were tried with cement and sand mixed in the proportions of 1 to 2, and 1 to 3, and gauged with water, and the percentage of reduction due to admixture with each other was found to vary very little from the above; although the percentage of reduction due to admixture with water was greater in proportion to the larger volumes of sand.

TABLE 10.—PROPORTIONATE VOLUME and WEIGHT[1] of MATERIALS which should
and as ASCERTAINED by the MODE of

Number of Concrete corresponding with Number of Experiment in Table 8.	Nature and Dimensions of Aggregates.	Volume of Interstices in Percentage of Volume of Aggregates.	Proportion of Volume of Dry Cement and Sand to Volume of Aggregates.
I.	Welsh limestone broken by Blake's stone-crusher into irregular, flat, oblong pieces, most of which, if gauged the narrowest way, would pass through a 3-inch ring	50·90	1 to 1·20
II.	Gravel obtained from a bed near Northwich, screened free from sand, varying in size from small pebbles to pieces gauged by a 2½-inch ring	33·60	1 to 1·66
III.	Welsh limestone and gravel, well mixed in equal proportions, of the same dimensions as in experiments I. and II. .	34·00	1 to 1·65
IV.	Anglesea limestone (mason's shivers), varying in size from small gravel to pieces gauged by a 4-inch ring	48·00	1 to 1·25
V.	Runcorn red sandstone (large) broken by hand, varying in size from pieces gauged by a 4-inch ring to pieces gauged by an 8-inch ring	50·00	1 to 1·22
VI.	Runcorn red sandstone (small) broken by hand, varying in size from sand to pieces gauged by a 4-inch ring . . .	34·00	1 to 1·65
VII.	Runcorn red sandstone (large and small) mixed in equal proportions, of the same dimensions as in experiments V. and VI.	36·00	1 to 1·58

[1] The weights of the materials are given to facilitate the calculation of the cost (as in Tables 5
accurate for obtaining the approximate cost of concrete

COMPOSE DIFFERENT CONCRETES, as DEDUCED from the EXPERIMENTS (TABLE 8), MEASUREMENT PREVIOUSLY DESCRIBED.

Proportionate Volume and Weight[1] of Materials which should compose Concretes made with different Aggregates.						Volume of Cement to Volumes of other Materials composing Concretes.
Cement.		Sand.		Aggregates.		
Volume.	Weight.	Volume.	Weight.	Volume.	Weight.	
1	1	1·0	0·85	2·40	2·42	1 to 3·40
1	1	1·5	1·27	3·00	3·03	1 „ 4·50
1	1	2·0	1·70	3·60	3·63	1 „ 5·60
1	1	2·5	2·12	4·20	4·24	1 „ 6·70
1	1	3·0	2·55	4·80	4·84	1 „ 7·80
1	1	3·5	2·97	5·40	5·45	1 „ 8·90
1	1	1·0	0·85	3·32	3·92	1 to 4·32
1	1	1·5	1·27	4·15	4·90	1 „ 5·65
1	1	2·0	1·70	4·98	5·87	1 „ 6·98
1	1	2·5	2·12	5·81	6·85	1 „ 8·31
1	1	3·0	2·55	6·64	7·83	1 „ 9·64
1	1	3·5	2·97	7·47	8·81	1 „ 10·97
1	1	1·0	0·85	3·30	3·96	1 to 4·30
1	1	1·5	1·27	4·12	4·94	1 „ 5·62
1	1	2·0	1·70	4·95	5·94	1 „ 6·95
1	1	2·5	2·12	5·77	6·92	1 „ 8·27
1	1	3·0	2·55	6·60	7·92	1 „ 9·60
1	1	3·5	2·97	7·42	8·90	1 „ 10·92
1	1	1·0	0·85	2·50	2·40	1 to 3·50
1	1	1·5	1·27	3·12	3·00	1 „ 4·62
1	1	2·0	1·70	3·75	3·61	1 „ 5·75
1	1	2·5	2·12	4·37	4·20	1 „ 6·87
1	1	3·0	2·55	5·00	4·81	1 „ 8·00
1	1	3·5	2·97	5·62	5·41	1 „ 9·12
1	1	1·0	0·85	2·44	1·92	1 to 3·44
1	1	1·5	1·27	3·05	2·40	1 „ 4·55
1	1	2·0	1·70	3·66	2·88	1 „ 5·66
1	1	2·5	2·12	4·27	3·36	1 „ 6·77
1	1	3·0	2·55	4·88	3·84	1 „ 7·88
1	1	3·5	2·97	5·49	4·32	1 „ 8·99
1	1	1·0	0·85	3·30	3·23	1 to 4·30
1	1	1·5	1·27	4·12	4·03	1 „ 5·62
1	1	2·0	1·70	4·95	4·84	1 „ 6·95
1	1	2·5	2·12	5·77	5·65	1 „ 8·27
1	1	3·0	2·55	6·60	6·46	1 „ 9·60
1	1	3·5	2·97	7·42	7·26	1 „ 10·92
1	1	1·0	0·85	3·16	3·06	1 to 4·16
1	1	1·5	1·27	3·95	3·83	1 „ 5·45
1	1	2·0	1·70	4·74	4·60	1 „ 6·74
1	1	2·5	2·12	5·53	5·36	1 „ 8·03
1	1	3·0	2·55	6·32	6·13	1 „ 9·32
1	1	3·5	2·97	7·11	6·90	1 „ 10·61

and 6) of concretes which may be made from the same; and they may also prove sufficiently produced from similar materials in other localities.

No. 1586.—"Portland Cement Concrete in Arches, and E
Cement Mortar." By CHARLES COLSON, Assoc. Inst. C

THIS communication embraces a record of the mean result perimeents on the tensile strength of coarse and fine gauged neat, and with an admixture of sand.

The course adopted in carrying out the experiments was from each cargo of cement, after being discharged into bushel as delivered and 1 bushel screened through each following screens, viz., 34 × 34 = 1,156 meshes, and 58 × 58 meshes per square inch. A few samples were also screened 65 × 65 = 4,225 meshes per square inch. The proportio taken by measure, the weight being also taken as a inasmuch as it is possible, by allowing the cement or sand too rapidly or too slowly into the measure, to have a varying density, an error immediately detected if the pr of taking the weight is followed. Of the cement as deliv the works, the average percentage of coarse, when screened 1,156 meshes per square inch, was about 11 per cent.

Table 1 contains a summary of experiments, showing th breaking tensile strain on 2·25 square inches, and also per inch of sectional area after being one month in water; the of the strength of the fine cement, as compared with the and the ratio of strength when gauged neat, and when with sand in the proportions of 1 to 1 and 2 to 1. The show the advantages of extreme fineness. That it is nec under all circumstances to screen the cement to the ex degree of fineness, as adopted in these experiments, is means advanced; for special purposes, however, it might sirable to do so, and possibly to exceed it. It is, howeve mitted that, for all ordinary purposes, the degree of fi adopted in specifications should in no case be less than 1,000 m per square inch, and not that any given percentage o quantity should pass through, but that all should do so; that the weight of the cement of this degree of fineness sh not be less than 85 lbs. per cubic foot. The mean breaking te strain of the thirty-nine samples referred to in Table 1, seven days' immersion in water, gauged neat as delivered on works, was 665 lbs. on 2·25 square inches, = 295·50 lbs. square inch.

PORTLAND CEMEN
IN BEAMS, AND

PLATE 11.

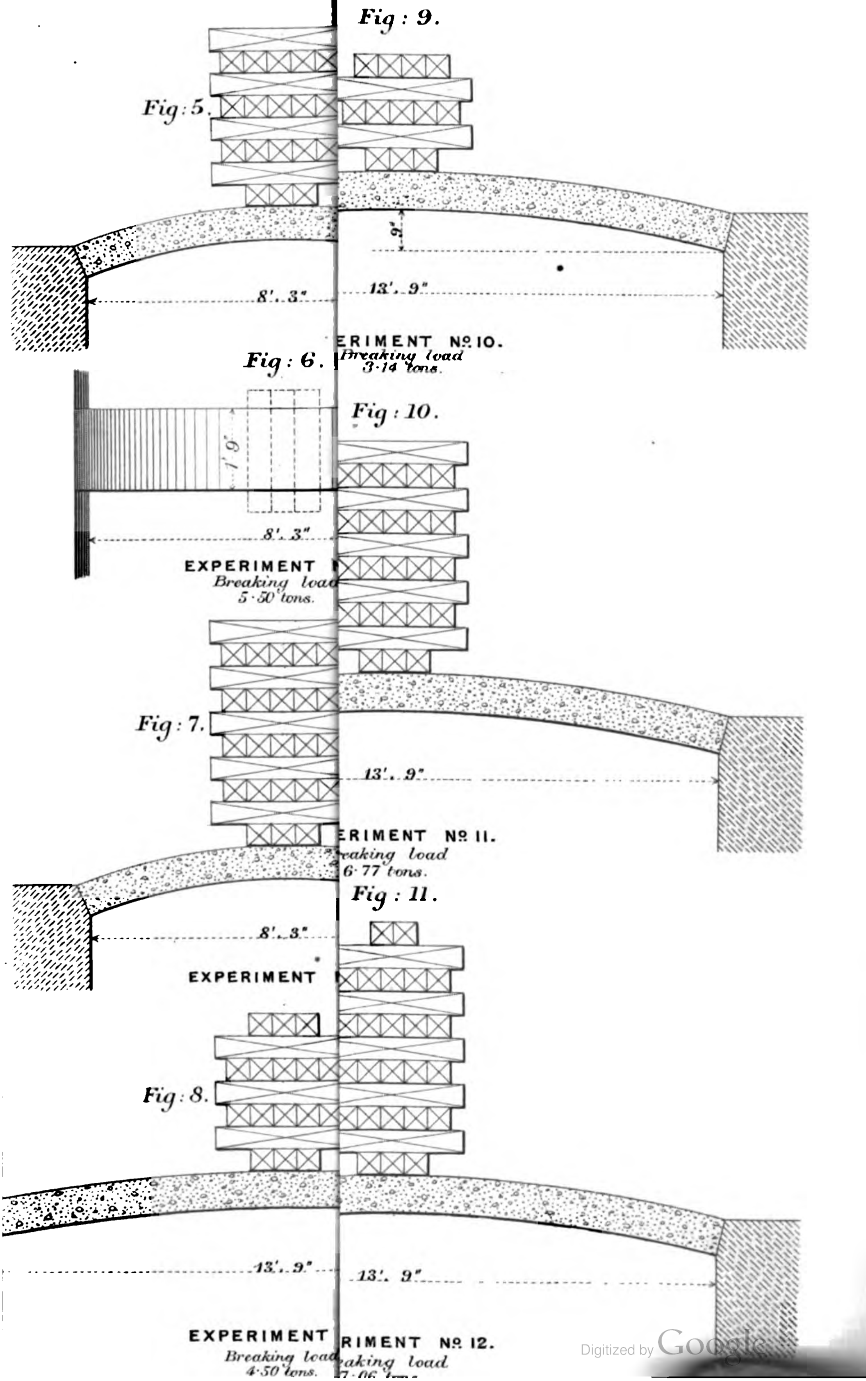


TABLE 1.—COMPARATIVE TENSILE STRENGTH of PORTLAND CEMENT ONE MONTH OLD, SCREENED and UNSCREENED, GAUGED NEAT and MIXED with SAND in the PROPORTIONS of 1 to 1 and 2 to 1.

Number of Samples Tested.	Neat Cement.								1 Sand to 1 Cement.								2 Sand to 2 Cement.							
	As Delivered.	Number of Tests.	Screened through 34 × 34 = 1,156 Meshes per sq. inch.	Number of Tests.	Screened through 58 × 58 = 3,364 Meshes per sq. inch.	Number of Tests.	Screened through 65 × 65 = 4,225 Meshes per sq. inch.	Number of Tests.	As Delivered.	Number of Tests.	Screened through 34 × 34 = 1,156 Meshes per sq. inch.	Number of Tests.	Screened through 58 × 58 = 3,364 Meshes per sq. inch.	Number of Tests.	Screened through 65 × 65 = 4,225 Meshes per sq. inch.	Number of Tests.	As Delivered.	Number of Tests.	Screened through 34 × 34 = 1,156 Meshes per sq. inch.	Number of Tests.	Screened through 58 × 58 = 3,364 Meshes per sq. inch.	Number of Tests.	Screened through 65 × 65 = 4,225 Meshes per sq. inch.	Number of Tests.
39	939·346	263	873·265	271	824·159	270	777·190	42	463·855	270	509·218	256	549·973	264	569·342	37	296·241	265	338·917	255	377·096	250	415·856[1]	28
	417·287		388·118		366·293		345·418		206·160		226·319		244·432		253·033		131·663		150·630		167·598		184·525[2]	
	1·00		0·929		0·877		0·827		1·00		1·097		1·183		1·227		1·00		1·144		1·272		1·403[3]	
	1·00		..		..		..		0·493		..		..		..		0·315		..		..		..[3]	
	..		1·00		..		..		..		0·585		..		..		..		0·388		..		..[3]	
	..		..		1·00		..		..		..		0·667		..		..		..		0·457		..[3]	
	..		..		..		1·00		..		..		..		0·732		..		..		..		0·535[3]	

[1] Mean breaking strain on 2·25 square inches. [2] Mean breaking strain per square inch. [3] Ratios.

Table 2 shows the relative weight of the same samples of cement as delivered, and when screened through the various descriptions of sieves used throughout these experiments.

TABLE 2.—COMPARATIVE WEIGHT of COARSE and FINE PORTLAND CEMENT.

Number of Samples Tested.	As Delivered.	Screened through 34 × 34 = 1156 Meshes per square inch.	Screened through 58 × 58 = 3364 Meshes per square inch.	Screened through 65 × 65 = 4225 Meshes per square inch.	Remarks.
40	112·15	104·850	98·300	94·750	lbs. per bushel.
	87·61	81·910	76·790	74·020	lbs. per cubic foot.
	1·00	0·934	0·876	0·844	Ratios.

The results shown in the Table bear upon a point before referred to,[1] namely, that the weight of Portland cement, taken alone, is no true indication of its quality. A weight of 104 lbs., compared with 112 lbs. per bushel, would be considered exceedingly low, and would be sufficient, if taken alone, to condemn the sample as not complying with the specification; whereas the cement might be of equal, if not superior, quality to a sample of greater weight, but being ground or screened finer, would give a less result, both when weighed and when tested neat.

Table 3 shows the comparative results of experiments on

TABLE 3.—COMPARATIVE BREAKING TENSILE STRAIN on PORTLAND CEMENT GAUGED NEAT, and MIXED with DIFFERENT MATERIALS in the PROPORTION of 1 to 1 TESTED when ONE MONTH OLD.

Number of Tests to each.	Neat.	Sand.	Crushed Granite.	Crushed Brick.	Crushed Portland Stone.	Remarks.
56	985·75	542·75	594·07	671·85	700·87	lbs. on 2·25 sq. in.
	438·11	241·22	264·03	298·60	311·50	lbs. per square inch.
	1·00	0·55	0·60	0·68	0·71	Ratios.

cement gauged neat, and mixed with porous and non-porous materials in the proportion of 1 to 1, sand and crushed granite

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xli., p. 127.

representing the non-porous, and crushed brick and Portland stone the porous. Exactly the same proportions by measure were taken of each. It was, however, necessary to increase the quantity of water in the case of the brick and Portland stone, in consequence of the porous nature of the materials. It will be seen that the resulting tensile strain obtained from the admixture of porous material considerably exceeded that obtained from the non-porous.

Comparative Tensile Strength of Grey Lime and Portland Cement Mortar.

The object of these experiments was to ascertain what proportions of Portland cement and sand would produce a mortar equal in strength and as convenient to work as grey lime mortar, the proportions ordinarily adopted for constructive purposes. The mortar was mixed to a workable consistency, equal, in fact, to the condition in which it would be used in the work. The mass was then moulded in the frames used for testing Portland cement, where it remained until sufficiently hard to admit of removal. At the expiration of six months the blocks were tested for tensile strength, the results being shown in Table 5, Nos. 1, 2, 3, and 4. With regard to the lime mortar (No. 1), the fractured blocks showed that induration, or the chemical action of setting, had penetrated only to the extent of from $\frac{1}{8}$ inch to $\frac{3}{16}$ inch, but in the majority of instances to only $\frac{1}{8}$ inch. The remainder of the area, although dry and moderately hard, had become so mainly from the evaporation of the moisture originally contained in the mass, and in no sense from the absorption of carbonic acid. It was possible, moreover, to crush it in the hand without any great exertion of force. The cement mortar, mixed in the proportions shown in No. 2, was of such a raw, harsh character, particularly that having the greater proportion of sand, that it would be practically impossible to use it in a satisfactory manner. In order, therefore, to render it somewhat more convenient for working, a small quantity of lime or yellow loam was added, thus rendering the mortar more plastic and tenacious. The results of further experiments (Nos. 3 and 4) show that the addition of lime and loam reduces the initial strength of cement mortar considerably, the reduction due to the addition of loam being more marked than by the addition of lime. The quantity of unslacked lime or loam, viz., $\frac{1}{12}$ the bulk of sand, was found to be as small a proportion as could be used, to give the necessary tenacity.

TABLE 4.—COMPARATIVE STRENGTH of GREY LIME and PORTLAND CEMENT MORTAR; also PORTLAND CEMENT MORTAR with the ADDITION of LIME and LOAM.

No. 1.—GREY LIME MORTAR.

No.	No. of Tests.	Proportions. Sand.	Lime.	Water.	Breaking Strain on 2·25 sq. ins. in lbs.	Breaking Strain per sq. inch in lbs.	Remarks.
1	17	2·00	1·00	1·33	61·06	27·13	Three samples.
2	27	2·00	1·00	1·33	106·07	47·09	Grey lime.
3	27	2·00	1·00	1·33	82·00	36·44	Water includes that required for slacking the lime.
Means	..	2·00	1·00	1·33	83·04	36·88	

No. 2.—PORTLAND CEMENT MORTAR.

No.	No. of Tests.	Proportions. Sand.	Cement.	Water.	Breaking Strain on 2·25 sq. ins. in lbs.	Breaking Strain per sq. inch in lbs.	Ratio compared with Lime Mortar.	Remarks.
1	15	6·00	1·00	1·25	233·53	103·79	2·81 to 1	Cement taken from bulk in store.
2	20	8·00	1·00	1·66	154·80	68·80	1·86 „ 1	
3	35	10·00	1·00	2·00	112·88	50·16	1·36 „ 1	

No. 3.—PORTLAND CEMENT and LIME MORTAR.

No.	No. of Tests.	Proportions. Sand.	Cement.	Lime.	Water.	Breaking Strain on 2·25 sq. ins. in lbs.	Breaking Strain per sq. inch in lbs.	Ratio as compared with Lime Mortar.	Ratio as compared with Cement Mortar.	Remarks.
1	70	6·00	1·00	0·50	1·50	165·31	73·47	2·00 to 1	0·70 to 1	Water includes that required for slacking the lime.
2	74	8·00	1·00	0·66	2·00	132·62	58·94	1·60 „ 1	0·85 „ 1	
3	85	10·00	1·00	0·83	2·50	95·27	42·34	1·14 „ 1	0·84 „ 1	

No. 4.—PORTLAND CEMENT and LOAM MORTAR.

No.	No. of Tests.	Proportions. Sand.	Cement.	Loam.	Water.	Breaking Strain on 2·25 sq. ins. in lbs.	Breaking Strain per sq. inch in lbs.	Ratio as compared with Lime Mortar.	Ratio as compared with Cement Mortar.	Remarks.
1	21	6·00	1·00	0·50	1·00	136·80	60·80	1·64 to 1	0·58 to 1	Yellow loam fresh dug and rather damp.
2	25	8·00	1·00	0·66	1·33	86·48	38·43	1·04 „ 1	0·55 „ 1	
3	19	10·00	1·00	0·83	2·00	64·50	28·66	0·77 „ 1	0·57 „ 1	

As regards the comparative adhesive power of these mortars, experiments made with the view of ascertaining the force required to separate bricks, joined with the several descriptions of mortar, were not altogether satisfactory, inasmuch as the appliances at hand were not sufficiently accurate and delicate to justify a ratio of comparison. It may, however, be stated that the general result went to prove that the adhesive power of mortar, mixed in the proportions of 8 of sand to 1 of cement, with the addition of loam, was superior to grey lime mortar mixed in the proportions of 2 of sand to 1 of lime.

Another point for consideration is the comparative cost of the different descriptions of mortar. Such estimates must, however, be received with caution, because difference of locality would exert a great influence upon the cost of production. The following statement, however, is a close approximation to the cost of the several descriptions of mortar; the charge for labour and water, and also the bulk of mortar produced, being in each case the mean results of experiments:—

TABLE 5.—STATEMENT of APPROXIMATE COST.

Description.	Proportions in cubic yards.				Cost of Materials.	Cost of Labour and Water.	Total Cost.	Produce of Mortar in cubic yards.	Cost per cubic yard.
	Portland Cement at 45s. 8d. per cubic yard.	Grey Lime at 14s. 6d. per cubic yard.	Loam at 2s. 9d. per cubic yard.	Sand at 2s. 9d. per cubic yard.					
					s.	s.	s.	c. yds.	s.
Grey lime mortar .	..	1·00	..	2·00	20·00	6·62	26·62	2·25	11·83
Portland cement mortar, No. 1	1·00	..	..	6·00	62·19	6·84	68·23	5·90	11·56
Ditto, „ 2	1·00	..	..	8·00	67·69	7·80	75·49	7·60	9·93
Ditto, „ 3	1·00	..	..	10·00	73·19	9·57	82·76	9·30	8·88
Portland cement & lime mortar, No. 1	1·00	0·50	..	6·00	69·44	8·68	78·12	6·40	12·20
Ditto, „ 2	1·00	0·66	..	8·00	77·27	11·20	88·46	8·25	10·72
Ditto, „ 3	1·00	0·83	..	10·00	85·22	13·78	99·00	10·15	9·75
Portland cement & loam mortar, No. 1	1·00	..	0·50	6·00	63·56	6·23	69·79	6·10	11·44
Ditto, „ 2	1·00	..	0·66	8·00	69·53	8·05	77·58	7·90	9·82
Ditto, „ 3	1·00	..	0·83	10·00	75·47	9·97	85·44	9·75	8·76

EXPERIMENTS ON THE STRENGTH OF PORTLAND CEMENT CONCRETE ARCHES AND BEAMS. (Plate 11.)

The object of these experiments was to ascertain, if possible, the relative supporting power of masses of concrete equal in quality and practically so in bulk, but differently disposed. It was also

desired to show the relative supporting power of arches of the same span, rise, and thickness at the crown, composed of porous and non-porous material respectively, such as shingle and broken bricks, but mixed in the same proportions. The proportions, in the experiments in which shingle was used, were 6 of screened harbour shingle, 3 of sand, and 1 of Portland cement. The proportion of sand was determined by the measurement of the quantity of water required to fill the interstices of the shingle when placed in a known cubic measure. In mixing the concrete as little water as possible was used consistent with thorough manipulation; and in depositing upon the centre great care was taken that there should be no horizontal beds or laminations, but that the whole should form a thoroughly homogeneous mass.

No. 1 experiment consisted simply of a beam of concrete, mixed in the proportions as before explained, 9 feet long, 1 foot 9 inches wide, and 9 inches deep; the distance between the supports was 8 feet 3 inches, with a bearing of 4½ inches at each end. Fourteen days after mixing the supports were removed, when the beam suddenly gave way near the centre. The fracture showed that the concrete was perfectly sound, there being no vacuities whatever to cause a diminution of effective sectional area.

No. 2 experiment consisted of a concrete beam similar in all respects to No. 1 (Plate 11, Fig. 1), being made in fact from the same mass and deposited at the same time. In consequence, however, of No. 1 having failed on the removal of the supports at fourteen days, the supports were not removed in this case till twenty-one days after mixing. At this interval the beam stood perfectly sound, and remained unsupported for a further period of seven days, when it was tested, and broke under a central load of 5 cwt.

No. 3 experiment consisted of a portion of No. 1 beam (Fig. 1) 4 feet 6 inches long, placed on supports 3 feet 9 inches apart (Fig. 2). This beam supported a load of 7·50 cwt. for seven days, when the load was increased to 13·75 cwt., under which load the beam failed twenty-eight days after mixing.

No. 4 (Fig. 3) consisted of a portion of No. 2 beam, 3 feet 9 inches clear of the supports, and tested immediately after No. 2, viz., at twenty-eight days after mixing. This beam failed under a load of 1·044 ton placed at the centre.

In each of these experiments the beams broke suddenly, without the least evidence, either by gradual cracking or otherwise, that the limit of load had been reached. One point, with regard to Nos. 3 and 4, deserving notice, is the difference in the load borne by

each before fracture took place, the interval of time being the same. It is possible that in the case of No. 3, which consisted of a part of No. 1 beam, the portion appropriated may have been slightly strained at the time of the first fracture on the removal of the supports at the expiration of fourteen days. The concrete for Nos. 1 and 2 beams having been mixed in one mass and deposited at the same time, there can be no doubt as to their being of the same quality.

The foregoing experiments being upon beams resting simply on vertical supports, it was desired to know what increase of resistance to fracture would be derived from the ends of the beam being blocked in such a manner as to secure perfect rigidity. Sufficient concrete was therefore mixed in the proportions before described to form two beams. One, No. 5, was formed with the ends resting on piers, as in the case of the previous experiments (Fig. 1). The second, No. 6, was formed between two counterforts of the wall, in which bearings 4½ inches deep had been cut (Fig. 4). After an interval of fourteen days the supports were removed, when No. 5 beam broke with its own weight in exactly the same way as No. 1. Having in view the first failure, additional precautions were taken in removing the supports, the folding wedges and bearers being all planed true in order to reduce the friction to a minimum. The circumstances attending the failure of these two beams being precisely the same lead to the conclusion that the strength of the concrete as used, at fourteen days' interval, was not sufficient to withstand the tensile strain at the centre due to its own weight. The supports were removed from No. 6 beam at the same time, no sign whatever of weakness being observed. After remaining unsupported for a further period of sixteen days the beam was tested by placing weights on the centre as before (Fig. 4). Under 0·25 ton a faint crack was observed at the centre through the whole width of the beam; with 0·635 ton it had increased as nearly as could be determined to half the depth, viz., 4½ inches, and opened to about $\frac{1}{16}$ inch at the lower surface. The full extent of the fracture probably exceeded this, although not apparent on the surface. No perceptible upward extension of the fracture could however be detected after the imposition of the weight last referred to. The load at the centre was ultimately increased to 1·292 ton, when the beam broke. This experiment shows the necessity of guarding against the possibility of lateral movement, in the slightest degree, in the supporting girders of a floor; in this case by so doing the supporting power of the beam was materially increased. It also shows that the mass within the

dotted line *a b c* (Fig. 4) adds nothing to the strength of the beam when confined at the ends, as proved by the crack appearing so soon after the commencement of the loading.

Experiments Nos. 7 and 8 were made to compare the gain in strength derived from a different disposition of the same bulk of concrete. The same proportions and dimensions were preserved, but the mass was deposited in the form of an arch with a rise of 9 inches at the centre (Fig. 5). The supports were removed from both arches at the expiration of sixteen days; there was, however, no necessity for their remaining supported for so long a time, as shown by subsequent experiments on arches of nearly double the span.

No. 7 (Figs. 5 and 6). Testing was commenced when the concrete was twenty-three days old. When loaded with 1·75 ton, a pig of iron, weighing 2·85 cwt., fell on one of the haunches, carrying away a portion, as shown in Figs. 5 and 6. The arch then stood for two days with a load of 3 tons on the centre. Under a load of 4·50 tons slight evidence of distress was observed at the crown, and with a load of 5·50 tons the arch failed by the complete crushing of the material at the centre.

No. 8 (Fig. 7). The testing of this arch commenced at twenty-eight days after mixing the concrete. When loaded with 5 tons, the testing was suspended for three days; it was then resumed, and with a load of 6·75 tons the arch failed. The slight indications of distress observed in the previous experiment appeared in this case, only immediately before the fracture of the arch.

No. 9 (Fig. 8). In this experiment the arch was 13 feet 9 inches between the abutments, 1 foot 9 inches wide, and 9 inches thick at the crown, with a rise of 9 inches. Exactly the same proportions of shingle, sand, and cement were used as before described. The centering was removed at seven days from the date of mixing the concrete, and the arch was tested at twenty-one days. A gauge was fixed in order to ascertain the amount of deflection due to the imposed load, which consisted of pig-iron ballast applied at the centre of the arch. With a load of 4 tons slight signs of distress were observed at the crown, when the gauge registered a deflection of $\frac{1}{8}$ inch. With an additional load of $\frac{1}{2}$ ton, = 4·50 tons, the arch suddenly failed, with no greater indication of distress than was previously observed. The greatest deflection registered was $\frac{3}{16}$ inch.

No. 10 (Fig. 9) consisted of an arch of the same span and dimensions, constructed and the centering removed on the same dates as No. 9, but tested after one month. With 3·14 tons at

the centre this arch failed, without even the slight warning observed in the previous experiments.

The concrete for the last two experiments was mixed in one mass as regards proportions of cement, sand, and shingle. No. 10 was, however, made much wetter than No. 9, to ascertain the effect of an excess of water upon the concrete. Judging from the results of these experiments the effect was to materially reduce the strength of the concrete. This was also found to be the case when concrete blocks were subjected to compression in the hydraulic press. These blocks, 6 inches × 6 inches × 6 inches, were composed of the same proportions of cement, sand, and shingle as used in the arches. An equal number was mixed with a maximum and minimum of water and tested when six months old. The same effect was also observed when broken bricks and broken Portland stone were used for the cement. The results of these experiments are shown in Table 6, Nos. 1, 2, and 3.

TABLE 6.—COMPARATIVE STRENGTH of CONCRETE when MIXED with a MAXIMUM and MINIMUM of WATER.

Materials—Broken brick, harbour shingle, and Portland stone.

Proportions—	Broken brick or stone	2·00
	Sand	1·00
	Portland cement	0·33

Age when tested:—Six months.

No. 1.—BROKEN BRICK.

Number of Tests.	Size of Blocks.	Weight of Blocks in lbs.		Load in Tons.		Remarks.
		In Air.	In Salt Water.	At Commencement of Fracture.	At Completion of Fracture.	
Maximum of water.						
12	Inches. 6 × 6 × 6	16·20	8·00	24·00	25·08	All blocks kept in water till tested.
Minimum of water.						
11	6 × 6 × 6	16·27	8·31	27·11	28·09	Ratio 1·13 to 1.

No. 2.—PORTLAND STONE.

Number of Tests.	Size of Blocks.	In Air.	In Salt Water.	At Commencement of Fracture.	At Completion of Fracture.	Remarks.
Maximum of water.						
24	Inches. 6 × 6 × 6	17·39	9·22	19·91	21·67	Blocks kept in water till tested.
Minimum of water.						
24	6 × 6 × 6	16·97	8·97	22·52	23·51	Ratio 1·13 to 1.

No. 3.—HARBOUR SHINGLE.

Number of Tests.	Size of Blocks.	In Air.	In Salt Water.	At Commencement of Fracture.	At Completion of Fracture.	Remarks.
Maximum of water.						
24	Inches. 6 × 6 × 6	18·05	10·07	17·17	19·69	Blocks kept in water till tested.
Minimum of water.						
14	6 × 6 × 6	17·61	9·79	22·83	25·09	Ratio 1·33 to 1.

NOTE.—The above ratios refer to the first appearance of weakness as being in fact the measure of the strength of the concrete.

In all the foregoing experiments ordinary shingle dredged from Portsmouth harbour, water-worn and rounded to a great extent, was employed. Although the expression, crushing of the concrete, has been used in describing the results observed, it is to be understood, not that the stone or shingle in the mass was crushed or fractured, but that the strain to which it had been subjected had in all cases overcome the adhesive power of the cementing material for the rounded, smooth, and non-absorbent surface of the shingle.

The concrete for Nos. 11 and 12 experiments was composed of broken bricks, mixed in the same proportions as used for the shingle arches, viz., 2 to 1 of broken brick and sand and 3 to 1 of sand and cement; before mixing, the broken material was well damped. The necessary bulk of concrete for both arches was mixed in one mass and deposited in position at the same time. The centerings were removed in both cases seven days after mixing, and the arches tested at twenty-eight days. In No. 11 experiment (Fig. 10), slight evidence of distress at the crown became apparent under a load of 6 tons on the centre, and with 6·77 tons the arch failed.

No. 12 (Fig. 11) was tested on the same day as No. 11 arch, and supported a load of 6·50 tons before any indication of distress was observed; the load was then gradually increased to 7·064 tons, under which the arch failed.

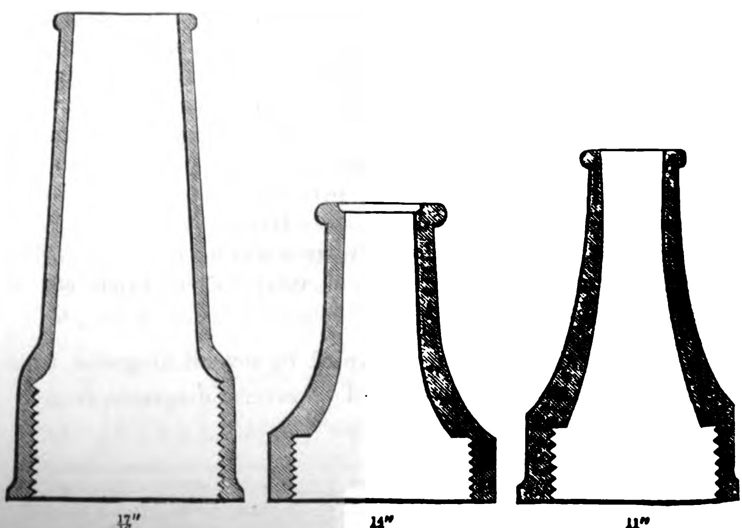
The superior strength of the arches in the last two experiments is evidently due to the more absorbent and angular character of the material. The appearance of the fractures in the two cases, *i.e.*, shingle and broken brick, showed a marked difference. In the first case, the strain destroyed the adhesive power existing between the shingle and the matrix; in no instance was a stone observed to be fractured, the casts being, as a rule, clearly defined in the cement. In the second case, the superior adhesive power existing between the broken brick and cement matrix was manifest; in but few instances had the cement left the surface of the brick, the general characteristic being that of complete disintegration of both brick and matrix.

This communication is accompanied by several diagrams, from which Plate 11 has been compiled.

No. 1576.—"Experiments on the Heights, &c., of Jets from the Hydrants of the Kingston Waterworks, Jamaica." By Felix Target, Assoc. Inst. C.E.

Numerous experiments were made with nozzles of various sizes and different lengths of hose, attached to hydrants on the street mains, which mains were of varying diameter. The accompanying table (see pp. 278, 279) gives the results of some of the experiments, those cases best suited for comparison having been selected. The height of the jet was measured from the outlet at the nozzle to the upper part of the curved spray described by the jet. The copper hand-pipe, 4 feet in length, was always held breast-high, with the nozzle 5 feet to 6 feet off the ground. The leathern hose was of the kind ordinarily used in London, $2\frac{1}{2}$ inches in diameter and in lengths of 40 feet. The hydrants and stand pipes were Bateman and Moore's. The mains were nearly new, and were coated inside with Dr. Angus Smith's preparation. The draught of water for the town for twenty-four hours was equal to 1,266,600 gallons, the maximum per hour being 93,000 gallons. During the time the experiments were carried on the draught was 45,000 gallons per hour, which is the average night consumption. The experiments were made in the early morning in a still atmosphere.

The accompanying figures show the forms of three of the



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nozzles. Up to the highest pressures the $1\frac{7}{8}$-inch nozzle threw a much more compact jet, with less spray, than either the $1\frac{4}{8}$-inch or the $1\frac{1}{8}$-inch nozzle, the smaller of which occasioned the greatest spray. The heights are only correct within a few inches, as the jets slightly varied during the time of the experiments, notwithstanding that the pressure gauge, which was used to ascertain the head of water, remained nearly steady.

From these experiments it is difficult to arrive at any correct law, or formula, for calculating both the height and the delivery of water from jets in a town. It is evident, however, that with high pressures, although the 2-inch mains are large enough to furnish an ample and constant supply to forty houses, each drawing from 200 gallons to 500 gallons per day, yet they are undoubtedly too small for fire purposes without the aid of a fire engine.

The 4-inch mains gave results nearly equal to the 12-inch mains with an effective head of 155 feet. Taking height and quantity into consideration the $1\frac{4}{8}$-inch nozzle with the higher pressures appeared to give the best results.

RESULTS of EXPERIMENTS on the HEIGHTS of JETS, DELIVERY of WATER, &c., at the KINGSTON WATERWORKS, JAMAICA.

1 Number of Experiment.	2 Size of Nozzle in Inches.	3 Height of Jet in Feet.	4 Number of Gallons per Minute.	5 Head in Feet at Hydrant.	6 Length of Main in Yards.
No. 1.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$	20½ 34½	92 55	53½ ..	1,083 of 21-inch + 50 of 4-inch.
Ditto, with three lengths of hose .	$\frac{17}{16}$ $\frac{14}{16}$	18 29½	.. ..	.. ..	
No. 4.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	38 44½ 44	122 73 66	92	1,585 of 21-inch +133 of 12-inch.
No. 5.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	9 25 27	66 55 47	92	1,585 of 21-inch +183 of 12-inch + 66 of 2-inch.
No. 6.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	55 68 77	138 94 73	122·4	1,585 of 21-inch +600 of 12-inch +116 of 4-inch.
Ditto, with three lengths of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	48½ 62 66	.. .. ..	.. .. ..	
Ditto, with six lengths of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	26½ 51½ 62	100 82 69	.. .. ..	
No. 7.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	7 24 27	52 47 ..	122·4	1,585 of 21-inch +600 of 12-inch +166 of 4-inch + 20 of 2-inch.
No. 9.—With one length of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	11½ 32 32	60 47 ..	106	1,585 of 21-inch +266 of 12-inch + 60 of 4-inch +100 of 2-inch.
Ditto, with three lengths of hose .	$\frac{17}{16}$ $\frac{14}{16}$ $\frac{11}{16}$	12 29 32	.. .. ..	.. .. ..	

RESULTS of EXPERIMENTS on the HEIGHTS of JETS, &c.—*continued.*

1 Number of Experiment.	2 Size of Nozzle in Inches.	3 Height of Jet in Feet.	4 Number of Gallons per Minute.	5 Head in Feet at Hydrant.	6 Length of Main in Yards.
No. 13.—With one length of hose .	$\frac{17}{16}$	58	136	156	1,585 of 21-inch
	$\frac{14}{16}$	85	130	..	+1,266 of 12-inch
	$\frac{11}{16}$	84	94	..	+116 of 4-inch.
Ditto, with three lengths of hose .	$\frac{17}{16}$	48	180	..	
	$\frac{14}{16}$	64	132	..	
	$\frac{11}{16}$	62	94	..	
Ditto, with five lengths of hose .	$\frac{17}{16}$	41	143	..	
	$\frac{14}{16}$	55	132	..	
	$\frac{11}{16}$	62	103	..	
No. 14.—With one length of hose .	$\frac{17}{16}$	15½	73	154¾	1,585 of 21-inch
	$\frac{14}{16}$	28	73	..	+1,266 of 12-inch
	$\frac{11}{16}$	35	60	..	+ 70 of 4-inch
					+ 87 of 2-inch.
Ditto, with three lengths of hose .	$\frac{17}{16}$	18½	78	..	
	$\frac{14}{16}$	29	68	..	
	$\frac{11}{16}$	46	66	..	
Ditto, with five lengths of hose .	$\frac{17}{16}$	14½	70	..	
	$\frac{14}{16}$	26	66	..	
	$\frac{11}{16}$	35½	55	..	
No. 20.—No hose. Direct from 2-inch main	$\frac{17}{16}$	10½	64	157	1,585 of 21-inch
	$\frac{14}{16}$	22½	55	..	+1,050 of 12-inch
	$\frac{11}{16}$	37	60	..	+ 125 of 4-inch
					+ 111 of 2-inch.

MEMOIRS OF DECEASED MEMBERS.

Mr. EDWARD DIXON, son of the late Mr. John Dixon, colliery proprietor, of Cockfield, in the county of Durham, was born there on the 13th of June, 1809. He was educated at Ackworth School, and began his engineering career on the Liverpool and Manchester railway, under his brother John, when employed by Mr. George Stephenson to superintend the construction of the line across Chat Moss. A few years subsequently he was engaged near Birmingham, under Mr. Robert Stephenson, who was then constructing the London and Birmingham railway. He next accepted the appointment of Resident Engineer, under Mr. Joseph Locke, on the London and Southampton railway, and during its construction resided, first at Wandsworth, and later, at St. Cross, near Winchester. After the completion of the railway he became a partner of the Messrs. Twynam, seed crushers at Northam, Southampton. He had not been there long before he again became occupied, under Mr. Robert Stephenson, on surveys for projected lines of railway in Buckinghamshire, Warwickshire, and the adjoining counties; and then in superintending the construction of the Rugby and Leamington railway, the Bletchley and Oxford, the Leamington and Coventry, and the Nuneaton and Coventry railways. Before the completion of the last-mentioned railways he acted as Resident Engineer of the London and Birmingham railway, during the illness of Mr. Robert Benson Dockray, at which time he resided in London. When no hope was entertained of Mr. Dockray's recovery the appointment was offered to Mr Dixon, but was declined by him. He returned to Southampton, where, in conjunction with Mr. Thomas Cardus, the railway contractor, he acquired the remaining interests of Messrs. Twynam's business, and, as head of the firm of Dixon and Cardus, he was for many years employed in business pursuits. He also took an active part in the welfare and politics of Southampton, was elected President of the Chamber of Commerce, and was made a Justice of the Peace of the borough. He was one of the founders and original directors of the Union Steam Ship Company, for many years the only Mail Service to the Cape of Good Hope and South Africa, and now one of the most flourishing companies whose ships sail from Southampton. After some years' failing health he

retired from business in October 1873, a complete invalid, and went to reside in Wandsworth. He died, after a short illness, on the 18th of November, 1877. He was elected a Member of the Institution of Civil Engineers on the 1st of February, 1842.

MR. RICHARD STUART NORRIS was born at Bolton-le-Moors in 1812, and was educated at the grammar school of that town and at a private academy. At the age of fifteen he was articled to Mr. W. S. Hall, an architect and surveyor, in Warrington. During the period of his pupilage the railway system was inaugurated, and the late Mr. Joseph Locke, being in want of assistants to carry out the works of the Grand Junction railway, requested Mr. Hall, who had been a fellow pupil with Mr. Locke, to transfer Mr. Norris to his service; this having been arranged, in 1830 his connection with the railway company commenced, in whose service he remained for thirty-two years. His great energy and ability soon attracted the attention of Mr. Locke, who entrusted him with many important duties, including the survey of portions of the railway from Warrington to Birmingham. In 1836 he was chief draughtsman in Mr. Locke's office at Covent Garden, Liverpool, at which time the entire staff of the Grand Junction railway consisted of the secretary, two clerks, and an office boy, in addition to the engineering staff. Upon the amalgamation of the Liverpool and Manchester, Grand Junction, and London and Birmingham railway companies, into the London and North-Western railway company, he was appointed engineer for the Northern Division, his office being at first at Warrington, subsequently removed to Liverpool. At one period of his career he combined the duties of traffic superintendent with those of engineer; but later on, it being found advisable to separate those offices, he confined his attention exclusively to his duties as engineer. Having under his charge the whole of the London and North-Western system north of Stafford, and extending to Shrewsbury, Liverpool, Manchester, Bolton, and Leeds, the works he had to execute were numerous, although none of them were individually of very great extent. They consisted principally of rebuilding and enlarging stations and bridges, and the maintenance of the permanent way. He was remarkable for energy and promptness of action when any accident which unfortunately occurred required his presence. In spite of numerous engagements he found time to exercise his inventive powers, and took out several patents, one

of which was for a travelling hot-blast cupola for the rapid melting of metal: this patent was eventually sold to a Scotch railway company, but the use of it was given to the Government during the war with Russia, making red-hot shot, which it did with great effect. In 1852 he was nominated engineer to the Grand Trunk railway of Canada, but declined the appointment, owing to a reluctance to sever his connection with the London and North-Western railway. On this decision being made known, the working staff under his control presented him with a silver vase and a purse containing £100, as a token of their satisfaction at his remaining with them. When Mr. Norris retired from the post of passenger superintendent, the station-masters and other officials of that department gave him a massive tea and coffee service in silver, together with a handsome illuminated address. At the expiration of twenty-five years' service the directors presented him with the sum of £1,000, as a token of their appreciation of the valuable assistance derived from his exertions. In 1862 he retired from the service of the company, to whose interests he had devoted the best years of his life. His energetic mind would not however permit him to remain idle. He accordingly invested money in some collieries, and took an active part in their management and development. In order to be nearer to this new sphere of action, he left Liverpool and settled at Kenyon Junction, on the Liverpool and Manchester railway, where from his window he could see that line and the Kenyon and Bolton railway, both of which had been for so many years under his charge. In his retirement he continued to take delight in professional matters, and invented a manual coal-cutting machine, which has been tried with a considerable amount of success. He also acted as engineer for a proposed narrow-gauge railway from Buxton to Sheffield, through a most difficult country, but the scheme was bought up by the existing railway interests. In 1874 and 1875 he successfully carried through Parliament two Bills for the construction of the Wigan Junction railway, a most important undertaking, and one in which he took a deep interest. The preparation of the contract plans for this railway was his last professional work. In 1876, as soon as the contract was let, his failing health compelled him to relinquish all work. A few years before his death he received severe injuries from a fall while salmon fishing in Scotland, a sport of which he was passionately fond; and he never fully recovered from the effects of this accident. He died at Kenyon on the 26th of January, 1878, and was interred at Deane Church, near Bolton-ˋors. He was one of the few remaining links between the

former and present generation of engineers, having been an intimate associate of the Stephensons, Locke, and other founders of the railway system. As a man he was remarkable for his energy, good-nature, and sociability, while his charity was unbounded. Nothing gave him greater pleasure than to assist his younger professional brethren, both with advice and with material help; and probably no man was ever more beloved by his subordinates. Mr. Norris was elected a Member of the Institution on the 7th of March, 1854.

CAPTAIN SAMUEL JOHN DUNLOP was born at Portsmouth on the 11th of June, 1845. He was the second son of Lieutenant-Colonel Dunlop, late Paymaster of the 47th Regiment, and was educated at the Royal Military College, Sandhurst; from which he passed out first of his "batch," and was appointed to an Ensigncy in the Royal Welsh Fusileers on the 8th of January, 1864. Arriving in India, and having in a short time passed the higher standard in Hindustan, he proceeded, in 1867, to the Thomason Civil Engineering College at Roorkee, where he studied till the end of 1869, taking the second place and carrying off the College certificate, the Thomason Gold Medal for the best design, and a prize for drawing. He now joined the Bengal Staff Corps, and was appointed an assistant engineer in the Department of Public Works in the Central Provinces. He subsequently was ordered to the Warora coal mines, arriving in September 1872. Such was the satisfaction he gave, that the Chief Commissioner of the Central Provinces stated, in his remarks on the Progress Report for 1872–73:—"The services of Lieutenant Dunlop in connection with these coal operations have been most meritorious, and it gives me much pleasure to notice them thus prominently." The Chief Engineer, the late Mr. T. W. Armstrong, M. Inst. C.E., in his report dated the 30th of July, 1873, wrote:—"Lieutenant Dunlop, from sheer hard work and incessant attention night and day to his duties, has, I regret to say, become so unwell as to be ordered away from Warora. This Assistant Engineer, for the past four months, was obliged each day (often three or four times) when sinking was in progress, and at night in addition, to go down the shaft and examine the excavation work, the pumps, and so on; and while on this duty to undergo the effect of 200 gallons of water a minute falling, 'shower-bath' like, down upon him from a height of from 40 to 75 feet. The temperature of the air in the shaft may be taken at 100° to 105°, and of the falling

water at 60°. I think when a servant of Government undergoes such hard work as this is, such a very unhealthy duty, and when, as the Lieutenant has frequently done, he gets up from his bed in the middle of the night, ill and weak, and goes down this shaft, subjecting himself to such severe work, his conduct should be specially noticed and receive particular commendation. In England it would be very trying employment; and in India, at Warora, it is perhaps the most unhealthy duty a European could be engaged upon." The result of all this hard and trying work was that Lieutenant Dunlop became seriously ill. He was invalided to England for two years, and arrived home early in 1874. Having, as he thought, somewhat recovered his health at the end of a year, he returned to duty in the Central Provinces, and was posted to the Hoshangabad District, where he continued till the time of his death. He was promoted to the grade of Executive Engineer in charge in March, 1876. Unceasing in the performance of his duty, he did not quit his post till death had laid hands on him. On the 7th of June, 1878, he went on sick leave to Pachmarhi Hills, to be with his wife and family, and died there on the 27th of the same month. His remains were interred on the following day with military honours by H. M. 33rd regiment, the funeral being attended by every one in the station. That he was universally beloved was testified by the attendance of the native clerks and others from all the Government offices. Lieutenant Dunlop became a Captain in the Staff Corps on the 8th of January, 1876. He was elected an Associate of the Institution of Civil Engineers on the 3rd of February, 1874.

MR. THOMAS BOUSTEAD NELSON, the eldest son of Mr. Thomas Nelson, of Carlisle and York, was born in January 1842. At an early age he was associated with and assisted his father, who, as a contractor, was carrying out extensive railway and other engineering works in the north of England. Amongst the leading works executed by them were the Castleton and Grosmont extension and the Whitby deviation branch, for the North-Eastern Railway Company; the York and Doncaster line through Selby; also a duplicate tunnel at Marsden, on the Manchester and Leeds line, for the London and North-Western railway company. Between 1868 and 1878 the firm likewise constructed for the North-Eastern railway company the Team Valley Extension line, the Knaresboro' and Boro' Bridge branch, the Leeds and Wetherby Junction, with the Station

approach lines, the Ferry Hill Station lines, and the Stockton and Castle Eden line. Several extensions were also carried out in Scotland for the Caledonian and Portpatrick railway companies, and in Cumberland for the Furness railway company, besides minor lines and other works. At the time of his death Mr. T. B. Nelson was engaged, on his own account, on an important contract, the widening of the main line between Bletchley and Roade, a distance of about 14 miles, for the London and North-Western railway company. The nature of his work during many years, involving exposure to all weathers, and his untiring zeal and energy in all matters connected with his business, told upon a constitution already previously shaken by a severe attack of rheumatic fever. In January, 1875, he took a trip to the Cape of Good Hope, returning to England in the summer greatly benefited by the change and rest. At the latter end of the month of June, 1878, he was visiting Ireland, and during his stay in Dublin was attacked with brain fever, which terminated fatally on the 2nd of July.

Mr. Nelson was respected by all with whom he was associated, for his straightforward manly qualities, and honesty of purpose. His energy of character and zeal enabled him successfully to accomplish a large amount of work, by which he gained an experience and a position rarely attained by so young a man. He was elected an Associate of the Institution on the 4th of February, 1868, and held a commission in the Engineer and Railway Volunteer Staff Corps, in which, as a junior member, he took considerable interest.

SECT. III.

ABSTRACTS OF PAPERS IN FOREIGN TRANSACTIONS AND PERIODICALS.

On the Displacement or Oscillation of the Bubble of Spirit-levels. By M. PLANTAMOUR.

(Comptes rendus, vol. lxxxvi., 24 June, 1878.)

This Paper gives the results of observations made in April and May last in a building near the Lake of Geneva.

Having laid a spirit-level horizontally on a very solid and firm table, M. Plantamour observed considerable oscillations of the bubble, not only from day to day, but even during the same day. Attributing these to a possible change in the condition of the table under varying atmospheric influences, he removed the level to the concrete floor of the building, but the same displacement occurred. Thinking now that these were due to the level itself, which was an old one, he substituted another of perfect construction, and which had been carefully tested. This he also laid on the floor, with its ends pointing east and west, and observations were taken hourly from 9 A.M. till midnight. The movements of the bubble were measured with great accuracy by a millimètre scale, and then plotted on sectional paper, where the abscissæ represented hours, and each millimètre of ordinate an angle of 0·35 second. Curves were thus obtained, which showed maximum diurnal elevations of the east end of the bubble, at about 5 P.M. on the 24th, 25th, and 26th April, and a gradual and progressive increase during this period, the amplitudes of the oscillations being respectively 8·4 seconds, 11·2 seconds, and 15·75 seconds.

In order to guard against any possible motion of the floor of the building, which had only been recently erected, and was near the lake, M. Plantamour removed the level to the cellar of his house (which had been built twenty years) and laid it on the floor, which consisted of tiles set in cement on a bed of concrete, 8 inches thick, and showed an almost constant temperature of 55° Fahr. But this level was only 4½ inches long, and it frequently happened that the bubble touched the end of the tube. A larger one, 7½ inches long, was therefore set up with all possible care: 1 millimètre of bubble displacement in this level represented an angle of 0·5 second.

These two levels were then laid 6½ feet apart, with their ends east and west. Oscillations of the bubble were again noticed, the

oscillations being more marked in the smaller than in the larger level. At this time independent observations were being made by M. Turretini in his laboratory 2 miles distant, and these gave precisely similar results, that is, an almost continuous elevation of the east end of the bubble, which M. Plantamour's large level showed to be as much as 17 seconds from the 3rd to the 6th of May. From this date to the 19th this eastward movement continued slowly, except on the 11th, when the bubble remained stationary.

Having now changed ends with the level, observations were continued from the 20th to the 31st May, and readings taken every three hours, from 9 A.M. to midnight. It was found that the eastward motion was arrested, and the daily oscillations were on the whole less regular, on the 21st, 25th, and 27th, sudden westward movements taking place, with returns to the east not less sudden.

From the 24th to the 29th May, observations were made with the smaller level placed north and south. In this position diurnal oscillations were perceptible, but the hours of maximum elevation were not the same as when it was placed east and west. On the 24th, 26th, 27th, and 28th May the maximum elevation of the north end occurred about noon, and on the 29th at midnight, while on the 25th the south end was elevated considerably at 6 P.M., and on the 26th at midnight. There had therefore been during this period a gradual elevation of the north end, interrupted by two sudden and remarkable elevations of the south end. M. Plantamour thus sums up the results of his observations:—"During certain periods there was a gradual elevation of the east end of the bubble, without any marked return to the west; at others a certain permanence of horizontality, and again oscillations as much from north to south as from east to west, but confined within tolerably small limits. I cannot account for these displacements. It would, however, be interesting to know if they have greater amplitude at the equator, and less in places nearer to the Pole than Geneva."

In the discussion which followed, M. d'Abbadie described the experiments he made in 1837 at Olinda, Brazil (Lat. 8° south), and in 1842 at Gondar and Saqa (Lat. 12° 36′ N. and 8° 12′ N.), which fully confirmed M. Plantamour's observations. In 1849 M. d'Abbadie continued his experiments in France, and obtained similar results; but thinking that the oscillations were possibly due to imperfection in the stands of the levels, he made fresh experiments in 1863 with specially constructed apparatus. The results of these he laid before the Association for the Advancement of Science at Bordeaux in 1872, and they all went to prove a displacement or movement of the bubble, or in other words, a change in the position of the vertical. A confirmation of these results was also obtained by M. Bouquet de la Grye at Campbell Island (Lat. 52° 34′ S.). It may reasonably be inferred, therefore, that these displacements occur everywhere. This concerns surveyors and astronomers materially, as it affects the determination of latitudes

and star declinations. M. d'Abbadie therefore recommends a continued investigation of the subject, and suggests that simultaneous observations should be made at distant points, as described by M. Plantamour at Geneva.

W. H. E.

Means adopted for Ranging the Centre Line of the St. Gothard Tunnel.

By C. Dolezalek, Section-Engineer of the St. Gothard Railway.

(Zeitschrift des Architekten- und Ingenieur-Vereins zu Hannover, vol. xxiv., cols. 185–193.)

The axis of the St. Gothard tunnel is a straight line about 9½ miles long, with rising gradients of 1 in 172 and 1 in 1,000 respectively from both ends towards its centre. At its extremities, viz. in Göschenen and Airolo, observatories were erected, distant 585 and 358 mètres respectively from the tunnel portals, in which were set up the transit instruments previously used in laying out the Mont Cenis tunnel.

The direction of the centre line is given from the observatory at night by a lamp placed over that point in it, inside the tunnel, which can be accurately observed directly, its ranging being thence produced by a theodolite as far as the heading permits. A direct observation as far into the tunnel as possible is therefore of the greatest importance, and to obtain this as well as longer station lengths for the ranging in the interior of the tunnel, the Author devised the contrivances which form the subject of this Paper.

In 1875, to allow the signal to be shown at the right moment to the observer, telegraphic communication was established between the tunnel portal and the observatory, in both of which batteries with Morse's instruments were set up, while, in the unfinished tunnel itself, a wire was joined on by the use of portable field telegraphs.

As petroleum lamps with a bright flame proved far superior to common miners' lamps for signalling at long distances, the Author constructed one with the brilliant-burner ("Rundbrenner") of Schuster and Baer of Berlin, which gave on trial 1·8 time better illumination than the ordinary petroleum lamp. This burner has a double set of pinions moving two half wicks with the greatest regularity, and is screwed on to a large metal vessel having what is called a "double-vase ring." As this allows petroleum to be afterwards poured in without unscrewing the wick-holder, the centering of the lamp (over any station) is not thrown out during the whole period of its use, since the openings in the two rings can be made to coincide or not, at will. The vessel is now levelled on a movable bronze tripod, the centres being made accurately to

coincide. This concentric position is in the first instance secured by the maker, but if thrown out at any time, the ring, on which the lamp rests, can be so set by small screws, moving in a circular slit, that the middle of the wick shall be concentric with the tripod, the ring in this case being eccentric to it. This adjustment, however, ought not to be necessary if the lamp is carefully handled. A cylindrical metal mirror is provided to intensify the brilliancy of the flame. This signal lamp surpassed all others in giving far longer station lengths under similar conditions; but it may even yet be advisable to devise apparatus for using the electric light in its place.

To diminish still more the delays and inaccuracies incident on such frequent settings-up of instrument and signal in the tunnel, the Author further constructed a stand applicable to either. It is in two parts, a top plate of metal resting on a larger circular one of wood to which three legs are attached. This top plate is separate from the lower one, though capable of being centred accurately with it under or over any required point inside the tunnel, such point being denoted by a notch on an iron cramp, which is driven into the ground. The weight (nearly 31 lbs.) of the metal plate ensures its steadiness, as its three pointed foot-screws work in small cups let into the wooden plate; by these it is levelled, and when the lamp is placed on it for use, it can be turned round and clamped in any direction.

Every station in the centre line was fixed by the mean of eight distinct settings-up of the lamp, by which all level and collimation errors were eliminated from the observations. To deduce this mean readily, the metal plate consists of a bronze plate sliding in a cast-iron frame and provided with a clamp and tangent screw. The centre of the bronze plate is given by a notch on either side, while to the two edges of the cast-iron frame strips of gummed paper are affixed, on which each observation is to be recorded by a pencil mark. To the mean of these marks the centre of the bronze plate is now set by the notch, and in order that it may necessarily be coincident with that of either lamp or theodolite, as each is successively set up upon the plate, three small grooves radiate from it at angles of 120°, in which are secured the feet of either instrument of whatever size. At the next station the used paper-strips are scraped off, and fresh ones affixed. A plummet and line are attached to the stand for centering purposes.

The advantages claimed for this stand lie in the remarkable speed of "setting-up," in the elimination of all possible errors in the operation, and in the ready insertion of the lamp upon it on the centre line; it is also easily carried about the tunnel packed in a chest. The wooden portion is only 1 mètre high besides the round wooden plate of 0·5 mètre outer, and 0·34 mètre inner, diameter. Lead weights are attached to the lower parts of the legs to keep them steady if accidentally pushed. Weights above 20 kilogrammes (44 lbs.) should be made up from smaller ones to facilitate their manipulation; and since all the material, instruments, &c., are

always forwarded from point to point in the tunnel on trollies, the transport of these lead weights offers no difficulty.

A light transit without vertical and horizontal circles, but with a powerful telescope magnifying thirty times, is advocated for ranging purposes inside the tunnel, and by its use greater rapidity in the work is anticipated.

E. H. C.

Experiments on the Strength of Cements and on the Bending of Bars. By H. Gollner.

(Technische Blätter, 1878, p. 1.)

These experiments were made with a special testing machine, lately erected for the Royal Technical High School at Prague. The first series was on the crushing strength of Portland cement, mixed in various proportions with sand, and tested partly in the form of cylinders, partly in that of cubes. The following table is condensed from those given in the Paper:—

Table 1.—Strength of Portland Cement.

Composition.	Cylinders.		Prisms.		Percentage of Strength, Prism to Cylinder.
	Age.	Crushing Load.	Age.	Crushing Load.	
	Days.	lbs. per sq. in.	Days.	lbs. per sq. in.	
1 part cement, 1 sand . .	378	47,600	312	33,600	72
1 „ „ 2 „ . .	387	48,400	312	30,600	62
1 „ „ 3 „ . .	311	25,700	310	22,000	85
1 „ „ 4 „ . .	120	11,400	310	8,000	70

In this table it is to be remarked that the cylinders in most cases stood the ultimate load for several hours before actually crushing, while the prisms crushed almost immediately. The result shows a great superiority in the cylindrical over the prismatic form, the mean strength of the latter being only 72 per cent. that of the former. This indicates that form has considerable influence on the resisting powers of cements; but probably the difference is in great measure due to its being much more difficult to bring the strain equally over all parts of a prismatic than of a cylindrical section.

The next series of experiments were made on the deflection of bars of various materials. The test pieces were 20 inches long,

loaded in the centre and held at the two ends. The following is a condensed table of results :—

TABLE 2.—DEFLECTION of VARIOUS MATERIALS.

Material.	Width of Specimen.	Thickness of Specimen.	Proof Load.	Proof Deflection.	Modulus of Elasticity.	Breaking Load.	Remarks.
	Inches.	Inches.	lbs.	Inches.		lbs.	
Beech . .	3·85	1·97	3,740	0·050	630,000	11,000	Fibres partly drawn out, partly broken sheet.
Cast iron. .	4·04	2·17	17,200	0·026	616,000	33,000	Metal apparently excellent.
Wrought iron (rolled) .	3·12	0·69	8,800	0·053	8,736,000	..	Very soft iron.
Wrought iron (forged) .	4·00	1·20	30,800	0·035	12,530,000	..	Hard and stiff iron.
Bessemer steel	4·00	1·01	31,400	0·041	13,895,000	..	0·55 per cent. carbon.
Cement: 1 cement . 1 sand . .	3·96	2·14	..	..	..	880	Coarse sand.
Cement: 1 cement . 2 sand . .	4·08	2·15	..	..	..	880	Average age, 394 days.

The "proof load" in the above table is that at which permanent set was first produced, or the limit of elasticity. The modulus of elasticity for loads below this limit is determined from the formula $E = \frac{1}{4}\frac{P\,l^3}{f\,b\,h^3}$, where P is the load in lbs., l, b, h, f the length, breadth, depth, and deflection respectively of the specimen, all in inches. It will be found on comparison that this modulus of elasticity for bending is much higher than the modulus of elasticity for extension or compression, both in the case of cast iron and of steel; and even for cast iron it is higher than that for extension, but about the same as that for compression. The very low result for rolled iron seems to have been due to the material being unduly soft.

W. R. B.

On the Influence of Welded Joints on the Ultimate Strength and Ductility of Bar-iron.

By Hr. STAMBKE.

(Wochenschrift des Vereines Deutscher Ingenieure, 1878, p. 219.)

The Author has made a series of experiments for the purpose of ascertaining the diminution of cohesive strength of bar-iron consequent upon welding.

The following descriptions were tested :—

			Millimètres diameter.	
Rivet iron	. .	from	23 to	28 (0·9 to 1·1 inch).
Round bars	. .	„	35 „	58 (1·37 to 2·28 inches).
Square „	. .	„	30	(1·18 inch).
Flat „	. .	„	12 × 32 „ 20 × 77	(0·47 × 1·26 to 0·78 × 3 inches).

Twenty-eight welded samples were tested, against the same number of unwelded pieces of equal section and quality. The average of the results was as follows :—

—	Ultimate Cohesive Strength.		Percentage of Original	
			Length.	Sectional Area.
	Kilogrammes per square Millimètre.	Tons per Square Inch.	Elongation.	Diminution of Sectional Area.
			Per Cent.	Per Cent.
Unwelded samples .	38·5	24·44	16·4	33·3
Welded „ .	31·9	20·21	7·3	10·9

From these results it would appear that the resistance to sudden strains and to the action of accumulated work is, as a rule, even more impaired by welding than the resistance to constant pressure. Consequently, whenever welding is indispensable, a considerably larger sectional area has to be adopted to ensure an equal degree of safety, as offered by unwelded bars.

A. H.

On the Determination of the Greatest Stress on the Diagonals and Verticals of Single-Trellis Girders.

(Die Eisenbahn, vol. ix., pp. 17–22.)

It is well known that to obtain the greatest stress in the members constituting the web of a detached girder, the movable load has to be assumed as only partially covering the girder. In practice the live load does not, as a rule, act directly on the main girders. The platform is carried by equidistant cross girders, which, in single-trellis girders, are attached to the verticals of the trellis, either at the top, the bottom, or at an intermediate point. The action of the live load is thus concentrated on equidistant single points of the main girders. Now, in computing the stress on the diagonals and verticals of the trellis, it is generally assumed that these points either receive the full amount of live load corresponding to one cross girder, or that they are entirely free from live load. Such an assumption simplifies the calculation, but is

not in accordance with the actual facts. For, with a uniformly-distributed load, a cross girder cannot receive its full amount of load without the next one already receiving at least one-half the full amount. The results furnished by the usual mode of calculation are therefore only approximate. The object of the present Paper is to show how, by making use of geometrical construction, the true maxima can be easily arrived at. The problem reduces itself to the determination of the point to which the load must have advanced within each bay to produce upon its diagonal and upright the greatest stress. This point once known, the corresponding shearing force is deduced, and from it the stresses. The Author treats the problem both graphically and analytically. In the latter case, after establishing the algebraical formulæ, he deduces from them a geometrical process, which, if desired, can serve as an independent check to the purely graphical method. Two examples of girders are illustrated in the Paper, and give a full elucidation of the described methods. One is of a girder with parallel booms, of 131 feet span, and 13 feet 1 inch in height, divided into ten equal bays by the trellis; the other is of a girder with curved booms, of the same span, but with a depth of 16 feet four inches at the centre, and divided also into ten equal bays by a single trellis.

A. O. B.

Raising the Frouard Station Road Bridge. By A. Picard.

(Annales des Ponts et Chaussées, 5th series, vol. xv., p. 592.)

The water-level of the portion of the Marne and Rhine Canal between Void and Jarville has to be raised in order that it may be incorporated in the East Canal, in course of construction, and have a depth of water of 6½ feet. This necessitates the raising of most of the bridges over this portion of the canal to obtain sufficient headway for the barges. The article gives a description of the raising of one of these bridges, consisting of a segmental masonry arch, of 32 feet 10 inches span, 21 feet 8 inches wide, and having a rise of 4¼ feet. The thickness of the masonry of the arch is 2 feet 10½ inches at the keystone, and 4 feet 11 inches at the springing. It was proposed to place the arch upon centering, to take it to pieces, and raising the centering 1 foot 2½ inches to rebuild the arch, slightly reducing its rise so as to diminish the amount of raising of the road and approaches. When, however, the work of demolition was commenced in July 1877, it was found that the masonry was so solid and good that it was decided merely to separate the arch from the abutments at the springing, and raise it bodily on the centering. The contact of the arch and centering was first made as perfect as possible, then the arch was cut at the springings and the ends supported. The centering deflected about 1½ inch at the crown when the weight of the arch

came on it, and a few slight fissures appeared in the arch. The centering, with the arch upon it, was then gradually raised by screw-jacks at the rate of about 1¼ inch per hour; the centering was lifted altogether 1 foot 5 inches, or 2½ inches more than the proposed rise, to allow for the deflection which had occurred in the first operation, and also for that which might occur on the final removal of the centres. The fissures in the arch were then carefully filled up with mortar, composed of equal parts of sand and Portland cement; the abutments were raised, the springing stones put on, and the ends of the arch joined to them, and eight days after the work had been completed the centering was removed, the deflection at the crown of the arch being only about one-twentieth of an inch. The great value of this method consists in shortening the time during which traffic on the canal has to be impeded, besides causing a notable saving in expense. If many bridges had to be similarly raised, it would be worth while to give great rigidity to the centres, and thus prevent any deflection of the centering and consequent injury to the arch.

L. V. H.

Headworks of the Ganges Canal. By Captain R. P. TICKELL, R.E., Executive Engineer, Northern Division, Ganges Canal.

(Professional Papers on Indian Engineering, Roorkee, vol. vii., p. 19.)

The Ganges Canal takes its supply from the Ganges near Bhímgoda. At this point the river has five channels, viz., the Dúdia, Nildára, Chiláwala, and new channel, and the main river. The head of the Dúdia channel is closed by a temporary bar which is only topped in high floods. The new channel is entirely closed by a crib and boulder dam; and the river is allowed to run in the main, Nildára, and Chiláwala channels only. The weirs form a nearly continuous line, passing across the bed of the river from high land to high land, the Dúdia and Chiláwala weirs forming the two flanks. The only break in the weirs is between Nildára and Chiláwala; but the nature of that part of the bed renders any protective works unnecessary for the present. To supply the canal, bunds are thrown across these channels, and the water is turned into what are known as Nos. 1, 2, and 3 supply channels, which all unite above Hurdwar, and form the Hurdwar supply channel which runs down to Maiápúr, where the required amount of water is passed into the Ganges Canal through the Maiápúr regulator, and the surplus is allowed to escape through the Maiápúr dam. The bunds across the Nildára and Chiláwala channels are made of boulders; those in the main river are constructed of cribs filled with boulders. All these bunds were carried away every rainy season, and had to be reconstructed during September and October. To obviate the inconvenience caused by this, it was determined to

build a permanent bar across the river at the general level of the cold weather bed. This plan had been tried with success on the Nildára channel, and has since been carried out on the Chilá-wala channel also. At this time the deep channel and main force of the river were on its left bank, and it was the unanimous opinion of the canal engineers that the river should be kept as much as possible on its right bank. It was, therefore, determined to commence the construction of the bar from the left bank, in hopes of driving the river towards the right. A length of 300 feet was built in 1864, and had the expected result. In 1874, the length of the bar had been increased to 590 feet; after the rains this length was found to be shingled up, and the force of the river passed beyond its right flank. In 1876 a further length of 290 feet was added, making the total length of the bar, from its left flank, 880 feet. It now became evident that the limit of such extensions of the bar from the left flank only had been reached; for the river was thrown with so much force on to the right bank that the destruction of Redan island between Nos. 1 and 2 channels was threatened. It was therefore determined that, before next rains, the bar must be completed across the Ganges, and that the nose of the island between Nos. 1 and 2 supply channels must be made strong enough to resist the full force of the river; and, to protect No. 1 channel from the increased scour, it was determined to carry the bar over the island and across No. 1 supply channel, and protect the flanks of the bar by wing-walls. Work was commenced at the end of the existing portion of the bar, which was carried completely across the Ganges and over No. 1 island.

The chief difficulty met with in the construction of this work was the management of the water, including the diversion or passing off of floods, freeing foundations from water, and keeping up the canal supply. No clay being available, fine shingle and sand had to be used to make the bunds water-tight. Plain shingle bunds were used in situations where there was little or no current, and no great depth of water; and boulder or crib bunds in places where the run was moderate or very strong, and the depth considerable. A mat of grass or fine branches woven together, carried at least 10 feet on the ground at the toe of the bund, was used to catch the small shingle and prevent percolation. Each crib is something over 10 feet wide. After having been put together on the foreshore of the river, each crib was taken by boat to be laid *in situ*, and being fastened by ropes to the one previously laid, it was lowered gradually into position, and filled with boulders. On commencing work in December 1876, a drain was dug into one of the channels of the river, and a length of about 150 feet of the foundations of the weir was enclosed by shingle bunds. This drain reduced the level of water on the foundation level to about 2·5 feet; and the shingle having been excavated to a depth of 1·5 foot, the square cribs, forming the framing of the weir, were countersunk into their places the remaining foot, and one by one filled with masonry, composed of boulders laid in equal parts of stone

lime, surkhé, and coarse clean shingle. In order to carry off the leakage, a new drain, 15 feet wide, was dug along the face line of the foundations, which, however, only lowered the surface to 5 feet above foundations, and a second bund had consequently to be made along the toe of the weir, forming a high level drain to carry off a portion of the leakage. A permanent sál wood tunnel, fitted with sluices, was also built under the weir, not only to carry off drainage from upstream of the weir, but to act as a main drain in case of any future works north of the present weir. By the aid of these arrangements, the crib and masonry work was connected with the Redan island, and the lower river wing-wall and the Redan revetments were built.

The next step was to close and drain No. 1 channel, and to take the supply for the canal through or over No. 1 bund, and divert it down supply channels Nos. 2 and 3. For this purpose a tunnel was constructed passing completely under the proposed diversion of the canal supply; 500 feet of No. 1 bund was lowered so as to allow the canal supply to pass over it, and a strong dam was constructed across the head of No. 1 channel in a similar manner to that before described. When the water had settled itself down to this regimen, the left flank of No. 1 bund was also lowered, and the water passing over this flank ran into the river, and not into the supply channels; therefore this portion of the work acted as a waste weir.

F. C. D.

Navigation of the Seine between Paris and Rouen.

(Notices sur les Modèles, Cartes et Dessins relatifs aux Travaux des Ponts et Chaussées, Exposition Universelle à Paris, 1878, p. 121.)

Descriptions are given in this article of improvements made in weirs on the Seine; also of a boat diving-bell to be used on the river, and of a fluviograph.

Weirs.—In the Poirée needle dam the only points of support are the sill of the weir, and the stays to the uprights. Consequently the size of the spars or needles increases rapidly with the height of the dam, and as they must be light enough to be handled by one man, there is a limit to their size. By the use, however, of horizontal boards for closing the weir between each upright, the thickness of timber for high dams can be considerably reduced. Thus a dam 13 feet high requires needles 8 inches square, whereas if the uprights are 3½ feet apart, horizontal boards 3 inches thick will suffice. Accordingly a sluice-gate has been designed of boards placed horizontally, and decreasing in thickness towards the top, united on the upper side by two sets of hinges so as to form a sort of jointed wooden shutter which slides between the flanges of the T irons of the uprights, and can be rolled up round a roller at the bottom of the shutter by chains turning on a winch above. The

shutter can be wound up or lowered to any distance above the sill; it can be worked with ease, and is more watertight than the needle dam. This sort of shutter has been used since May 1876 for closing several openings of the weir of Notre-Dame-de la-Garenne, where the top of the dam is 8½ feet above the sill.

To remedy the leakage in the Poirée needle dams, a sort of curtain has been devised, made of stout canvas laid upon laths, ¾ to 1⅛ inch wide and ⅜ inch thick, which is stretched across each bay on the upper side of the dam. This curtain is weighted at the bottom, and hooked on to the heads of the needles at the top. This watertight curtain has been adopted with success at the needle dams on the lower Seine. Its cost, with a roller at the bottom for winding up, and all complete, is about 9*s.* 5*d.* per square yard. In order to allow fish to get up or down rivers across which needle dams are erected, M. Caméré has substituted an open frame for the needles, in one or more openings, in a portion of the space between the uprights. These apertures can be either partially or entirely closed by jointed wooden shutters, and by leaving out boards at certain places it is possible to vary the position of the open passage.

Diving-bell Boat.—The object of this contrivance is to enable weirs to be repaired in flood-time without a cofferdam, and without hindrance to the navigation. It consists of an iron boat, 105 feet long and 24 feet wide, with an octagonal well in its centre. Inside the well a cylinder is placed, free to move up and down, and guided by rollers; it is formed of two concentric cylinders with diameters of 17 feet and 6½ feet respectively; it is unattached to the boat, and constitutes the diving-bell. The space between the cylinders is divided into watertight compartments into which water or compressed air can be introduced at pleasure, thus regulating the balance and immersion of the tube. The central shaft serves as the passage for men and materials. The air-lock is situated directly over the working chamber, which has a diameter of 17 feet at the bottom, and is from 10 to 16½ feet high. The boat is furnished with two tanks, having a total capacity of 78½ cubic yards, and by emptying them or filling them with water the draught of the boat can be varied from 2 feet 8 inches to 3 feet 11 inches, thus enabling the boat to pass over shallows in dry weather, or under low bridges in flood time. The cost of the diving-bell boat is £7,040, averaging £40 per ton for the structure, and £100 per ton for the machinery.

Fluviograph.—This instrument registers on a sheet of paper placed round a revolving cylinder or on a rotating disc, the variations in the level of the river, and the weir keeper is warned by an electric bell when the water level passes the prearranged limits. It thus affords a valuable continuous register of the state of the river, and leaves the weir-keeper free to attend to other duties when the flow of the river remains within certain limits, and warns him whenever the variation in the discharge renders it necessary to alter the area of the opening in the weir. To keep

the size of the apparatus within convenient limits, and to enable a large scale (one-third full size) to be adopted for the registration, it was decided that the record should be limited to the height of rise in the river at which the weir is completely opened. This also enables the instrument to be fixed on one of the abutments of the weir, from which it is removed as soon as the river reaches the coping. The dial fluviograph costs from £12 to £16, and the cylindrical one about £4 more.

L. V. H.

New and Simple Method of River Training.

By A. Geppert.

(Wochenschrift des Oest. Ingenieur- und Architekten-Vereines, vol. iii., p. 115.)

This mode of river training may be briefly described as consisting in the construction of a partition of sheet piling across the land submerged during floods, and so placed as to connect the two points of the river between which the course is to be straightened. On either side of the sheet piling and along the entire length, a wide trench is excavated, the bottom of which corresponds to the level of ordinary water line. The greater portion of the flood-water is thus made to flow off through this artificial channel, and in doing so tends to scour a new river bed in it.

The method has recently been applied in the river Lech, at "The Gechtle," and the Author gives an account of the works, and of the results obtained. The pile planking extends over a length of 1,867 feet, and intersects a large island which obstructs the direct course of the river. It is 4 feet high from the bottom of the ditch, and is formed of piles 15 feet 6 inches long, driven at intervals of 6 feet 3 inches, and planked on one side. The piles have a diameter of from $8\frac{1}{4}$ to $9\frac{1}{2}$ inches, and are shod in the ordinary way. The deal planks are $2\frac{1}{10}$ inches thick. The trench was made 6 feet 3 inches wide on each side of the line of piling, the soil of the excavation being simply thrown on the banks. The total cost of the work came to 9*s.* 6*d.* per lineal yard.

During the summer of 1877 various floods occurred in the Lech, which, although continuous, did not attain to any height. The last one alone reached the top of the piling and produced notable changes in the bed of the river, without, however, damaging in any way the planking. In a plan the Author shows the course of the river both before and after the flood. It gives a fair idea of the effect which the work has had on the course of the river. After the water subsided it was found that the river had taken a new bed all along the first third of the pile planking. Then, encountering higher ground in front, it had deviated to the left and had fallen back into the former bed of its left channel. On the other side of the planking, the right branch had also slightly

altered its course, and had been considerably reduced through silting. These first results were deemed so satisfactory, considering the small height to which the floods had risen, that it was decided to widen out the unaffected parts of the ditch before next year's floods. It is thought probable that the river will then scour out its bed along the entire length. As soon as this result will have been achieved, the Author will report again on the subject.

A. O. B.

Embankment of the Tidal Portion of the Seine.

(Notices sur les Modèles, Cartes et Dessins relatifs aux Travaux des Ponts et Chaussées, Exposition Universelle à Paris, 1878, p. 156.)

Before the lower thirty-seven miles of the Seine were embanked, the river was encumbered with shifting sandbanks which constantly altered the navigable channel. Below Quillebeuf the depth of water was only 14 feet at the highest tides, and $5\frac{3}{4}$ feet at high water neap tides; and above Quillebeuf the depth was in many places insignificant. Only small vessels of between 100 and 200 tons could navigate the river. Four days were occupied in the voyage from Rouen to the sea, a distance of 74 miles, for which the freights amounted to 7*s.* $10\frac{1}{2}$*d.* per ton. The embankment works were commenced in 1846, and the total cost up to the end of 1876 amounted to £678,060. The embankments were all made of random work (*pierres perdues*) with blocks of chalk quarried from the adjacent cliffs. Embankments rising above the highest tide-level have been constructed a little farther down than Tancarville on the right bank and to La Roque on the left bank; beyond these points submerged banks have been adopted. The banks which were injured, owing to the lowering of low water and by the bore in the Seine above Tancarville, have been protected by pitching. The works have produced a considerable deepening of the bed, in some places to the extent of 29 feet; the navigable channel has become more permanent, and large tracts of land have been reclaimed. The Meules bank is now the shallowest part of the river, but it has been lowered 10 feet by dredging, and there is never less than $15\frac{3}{4}$ feet of water over it at high-water neap tides. In 1861 the total tonnage of the vessels coming to and leaving Rouen during the year was 533,516 tons, and amounted to 732,456 tons in 1876; the tonnage of steamers coming up the Seine has increased from 109,607 tons in 1869 to 419,057 tons in 1876. The cost of freight has been reduced by one-half since 1867, and the transit from Rouen to the sea occupies one or at most two tides. Alluvial plains, 20,645 acres in extent, have taken the place of the shifting sands behind the embankments, and the value of these plains, when finally reclaimed, is estimated at £1,338,400.

Also, beyond the termination of the embankments large tracts of land have emerged from the sea, which become the property of the state.

L. V. H.

Movable Dam.

(Notices sur les Modèles, Cartes et Dessins relatifs aux Travaux des Ponts et Chaussées, Exposition Universelle à Paris, 1878, p. 46.)

The dam described is suitable for openings from 20 to 33 feet wide. It is divided into two openings by a central upright iron pile T-shaped in section. This pile turns on a horizontal axis at right angles to the stream, and is supported by a prop which moves on an axis turned obliquely to the line of the dam. The boards forming the dam rest against the central pile and recesses in the upper corners of the abutments 1 foot wide, the boards having only 4 inches of overlap against the pile and abutments. The dam can be gradually removed by drawing each board sideways along the recess of the abutment, by means of a chain attached to it, till it clears the central pile; it then floats down stream, and is laid on the bank. All the chains attached to each board of one opening are fastened to a ring fixed on the near abutment. When the dam is to be rapidly and completely removed, the prop is jerked away from the pile by a chain fastened near the upper end of the prop, and worked by a lever placed on the abutment. The prop falls clear of the pile, the pressure of the water makes the pile turn on its axis, and it falls flat on the bed of the river, and the boards floating down are arranged on the banks below. The pile, and, if necessary, the lower boards are replaced with the aid of one or two men in the river after the flood has passed, and the upper ones can be floated and guided into their places by the man on the footway of the dam. The sill of the dam is placed 8 inches above the apron of the dam, so that it may be kept clear from mud or gravel. The construction is very simple, cheap, easily worked, and readily repaired by country workmen. Four of these dams, placed on the rivers Vizézi and Moingt, cost on the average £100 apiece, the width, between the abutments, varying from 21 to 33 feet, and the height between 3 feet 7 inches and 6 feet 1 inch.

L. V. H.

The Floods of the River Po in the Nineteenth Century.

By Chevalier P. Gallizia, C.E.

(Giornale del Genio civile, an. xvi., pp. 3, 41, 125.)

The most remarkable floods to which the river Po has been subjected during the present century are those of the years 1801, 1807,

1810, 1812, 1823, 1825, 1827, 1829, 1839, 1840, 1841, 1846, 1855, 1857, 1863, 1864, 1868, 1872, and 1876, making nineteen floods in seventy-six years. The mean frequency has been one flood in each four years, with the exception of the fortunate decade 1813–1822. The above remarks apply to serious floods, as scarcely a year elapses without the river passing the warning signal in some of its lower branches. Since the hydrometer was erected at Pontelagoscuro in 1807, 262 rises above that mark have been recorded. The six years 1801, 1839, 1846, 1857, 1868, and 1872 are those which have witnessed the most abundant and disastrous floods.

Details are given of all these inundations, and profiles of those of the years 1839, 1840, 1841, 1846, and 1857. The most rapid in its rise was that of October 1857, which, between the 21st and the 25th of that month, rose from 24 to 25 feet at Ostiglia. It owed its origin to the rivers and torrents of the Apennines and of Piedmont, especially to the Tanaro and the Sesia, which then attained the greatest height on record. On the other hand, the floods of the Lombard lakes were moderate, the Lago di Como only rising 4·75 feet, and the Lago di Garda 2·70 feet. The Lago Maggiore, however, rose 13 feet, with an extraordinary rise of 4·30 feet in twenty-four hours on the 22nd, a phenomenon without parallel in that basin. But in the provinces of Cuneo and of Turin the rivers rose with unusual violence. The height measured at Ostiglia was 26·80 feet, and that above the warning signal at Pontelagoscuro 9·76 feet. In the upper provinces of Pavia, Lodi, and Piacenza, more than twenty breaches of the banks occurred, extending over a total length of nearly 1½ mile. Intelligence of the flood being transmitted along the course of the Po by telegraph, the precautions taken by Signor Lombardini maintained the integrity of the banks in the Mantuan and Ferrarese districts, although they were overflowed nearly 2 feet, as was also the celebrated *coronella dei Ronchi di Revere.*

The long list of disasters concludes with an account of the extraordinary flood of 1872, of which a full report was presented to the Italian Parliament by the Minister De Vincenzi. Consternation was spread along the whole course of the Po, and large tracts of country suffered serious disasters. The floods commenced in October, in which month thirty-three breaches opened in the banks of the river, and the consequent inundation was not entirely reduced till the April of the following year.

A tabular synopsis is given of the more serious breaches which have occurred, 214 in number, in the main banks of the Po, from the confluence of the Ticino to the sea, during the nineteenth century, divided according to their causes. These are stated to be overflow, corrosion, percolation, destruction of masonry, and unknown causes; 158 occurring from overflow. The district in question is divided into three tracts, and the total length of bank, on both sides of the river, for the entire distance, is 475½ miles. The paper next considers the actual condition of the embankments of the Po, and the necessity of promptly completing their systematic

reparation. It compares the hourly observations of the hydrometer in the last floods, and the respective diagrams, gives a general comparison of the highest floods of 1839, 1857, 1860, and 1872, and discusses the time occupied in the propagation of the floods, as shown by the curves on the diagram. It examines the maximum rises in various localities, and the general course of the floods, as well as the degree in which the highest levels marked on the different hydrometers would have been increased if no ruptures had occurred in the banks.

From the investigation of the question thus far, Signor Gallizia arrives at the conclusion that the progressive rise in the height of the floods is a subject for serious inquietude. The causes of this rise are diverse, the first mentioned being the destruction of forests on the hills. As to this, the writer considers that the passing of a good forest law is most desirable. The improvements in agriculture, leading to a more rapid drainage, and the progress of the work of embankment of the channel of the river, tend to increase the height of the flood levels. There remains for consideration the question of the protraction of the *foce*, or embanked channels of discharge, into the Adriatic, which it is desirable to continue as far as the depth of water in that sea will allow that work to be safely effected. From the scour thus to be secured, great relief to the river is anticipated.

A table is given of the mean monthly heights of the river at the seven principal hydrometers, as well as of the mean annual height, the level of the highest flood, and that of the lowest summer flow, or *magra*. The zero of the hydrometers generally corresponds to the level of the ordinary *magra;* but at the hydrometers of Pontelagoscuro and of Cavanelle the graduation of the scale refers to the warning signal, a height the attainment of which by the river is a presage of danger. On the whole it appears that the year in which the greatest recorded discharge occurred from the Po was the memorable epoch of 1872, with the exception that the flood of 1810 rose somewhat higher at Pontelagoscuro; the observations at other points being absent for that year. The lowest mean height was in 1828 at Pontelagoscuro, in 1854 at La Becca, in 1858 at Casal Maggiore and Cavanelle, in 1871 at Carossa, and in 1874 at Mezzanacorti and Ostiglia. The year 1862 was that in which the mean annual flow of the river was most closely in correspondence with the mean at every station of observation.

From the special examination of the observations at Pontelagoscuro it results that the mean height and outflow of the Po are continuously decreasing. This has previously been pointed out by the Inspector, Commander Natalini, with reference to the three periods, 1807–1825, 1826–1850, and 1851–1875, although the maxima and minima during those periods do not follow any accustomed line. The lowest *magra* was that of 1817, of 17·70 feet below the warning signal at Pontelagoscuro. But the Author is of opinion that accurate observations extend over too short a period to enable any one to define the main lines of the discharge of the

waters of the Po; and recommends the continuance of the hydrometric observations and their record with the utmost care.

F. R. C.

Blasting Operations in the Danube, at Nussdorf.

(Wochenschrift des Oest. Ingenieur- und Architekten-Vereines, vol. iii., pp. 127, 132.)

At the end of 1877 the Danube Training Commission decided on the removal of a series of dangerous reefs which hug the right shore of the river from opposite the entrance to the Danube canal to the "Kahlenbergerdörfel." It was proposed to deepen those points to a minimum of 10 feet 6 inches below the zero of water-mark, and this entailed the blasting and removal of some 23,500 cubic yards of rock.

The blast-holes varied in depth from 3 feet to 8 feet 4 inches, the greater number being about 5 feet. The depth of water in which they were bored was, as a rule, about 6 feet 6 inches. The boring was done partly by hand-drills, and partly by Schram and Mahler's steam rock-drill. Although the latter gave satisfactory results, inasmuch as it bored in two hours, and in 5 feet 9 inches of water, a mine 5 feet 8 inches deep, the Author thinks that during the cold season a rock-drill worked by compressed air will be preferable to one worked by steam, as a certain amount of condensation is then unavoidable. The drills were placed on a floating platform, formed by grouping together in a hollow rectangle twelve of the ordinary pontoons used by military engineers. The whole structure was firmly anchored in the river, besides being made fast to the neighbouring shore. Each mine received 0·74 lb. of dynamite per foot of depth of bore-hole. These strong charges completely disintegrated and shattered the rock around the hole to a distance equal to about one and a half times its depth. After clearing away the débris some further blasting was required in various places, and, to that effect, crevices produced by the first discharges were very effectively made use of. The mines were fired from the shore by means of electric fuses, the pontoon staging having previously been moved slightly up the river. On an average 0·7 lb. of dynamite shattered 1 cubic yard of rock, and the mean effect of one single mine amounted to 7·56 cubic yards. The actual cost of the work was 14*s.* 5*d.* per cubic yard of rock blasted and removed. This price, however, cannot be taken as a criterion, for the greater number of the men employed in the work were lent by the military engineering staff, as also the pontoons and some of the other appliances. The expenses for plant and labour enter therefore at a very low figure in above price. The Author estimates that, under ordinary circumstances, the same work could not be done for less than 18*s.* 6*d.* per cubic yard.

A. O. B.

Improvement and Irrigation of the Plain of Forez.

(Notices sur les Modèles, Cartes et Dessins relatifs aux Travaux des Ponts et Chaussées, Exposition Universelle à Paris, 1878, p. 36.)

The Forez plain, 240 square miles in extent, is intersected by the Loire, and bounded on the east by the Beaujolais mountains and on the west by the Forez chain. The small fall of the land, the choking up of the watercourses with soil and gravel, and the impermeability of the clay substratum have always rendered the plain unwholesome and barren. This state of things has, moreover, been aggravated by the formation of numerous ponds, used alternately for two consecutive years for rearing fish, and the water being then let off for the two following years for growing crops. One-tenth of the plain was covered with these ponds, or with natural marshes. The remedies proposed for these evils were cleaning out the watercourses, opening new drains, abolishing the ponds, and establishing a regular system of irrigation. The works are in course of execution on that part of the plain lying upon the left bank of the Loire above its junction with the Lignon; it is 74,130 acres in extent, and is traversed by two principal tributaries, the Mare and the Vizézi, whose basins have been separately dealt with. The works in the Mare basin for draining 32,900 acres, comprising cleaning out old watercourses and the formation of new drains for a distance of 69½ miles, and the suppression of ponds, are nearly completed at a cost of £21,600. The landowners contributed half, the state one-third, and the department one-sixth, of the cost. The cases of fever have been much diminished both in number and severity; the fertility of the land has greatly increased, and a considerable reduction in the time of duration of floods has been produced. The increase in the value of the land is reckoned at 25 per cent., amounting to more than eight times the cost of the works.

Similar works are in progress over 20,300 acres of the Vizézi basin at an estimated cost of £11,000. The Forez canal with branches and drains is being constructed for the irrigation of 64,250 acres of this portion of the plain. A minimum supply of water, obtained from the Loire, of 177 cubic feet per second, capable of being increased to 460 cubic feet, is provided for. The two first sections of the canal, 5 miles and 3¾ miles in length respectively, cost £5 2*s.* per lineal yard, and the third section, 1½ mile long, cost £5 6*s.* per lineal yard. The canal is being carried out by the department of the Loire, the state having furnished a grant of one-fourth of the cost: the department levies a rate of 11*s.* 4*d.* per acre irrigated, and is also allowed to supply water for mills and other purposes. The increase in value of the land is reckoned at £48 11*s.* per acre, and will amount to £960,000 when the works, costing £280,000, are completed.

L. V. H.

The Waterworks at Puerto de Santa Maria.

(Revista de Obras públicas, Madrid, May 15th to July 1st, 1878.)

The water with which this town is supplied is collected by means of a number of underground galleries driven into the side of a porous and spongy hill, composed of chalk and sandstone, called the Sierra de San Cristobal, at a distance of about 4 miles from Puerto de Santa Maria. The combined length of these galleries is about 840 yards, and, by excavating between them, an underground reservoir has been formed with an area of about 3,400 square yards, in which an average depth of a little more than 1 foot of water is collected. Some parts of this reservoir are lined with large blocks of ashlar, whilst others have been left with only the natural rock to support them, but more than a century has elapsed since the work was executed; in many places the sides have given way, and fears are entertained that this may continue on a larger scale.

The water is drawn off and allowed to settle in a second reservoir, whence it flows into a covered stone channel, and, after receiving on the way some trifling additions of water from other sources, is conducted to a small vault about 8 feet 6 inches square and 3 feet deep, which is the point of local distribution in the outskirts of the town. The difference of level between the two extremities of the main channel is only 18 inches, which allows of a very insufficient fall; and attempts have been made to raise the level of the water at the source by constructing stone dams at the outlet from the hill, but without success, for, owing to the porous nature of the soil, if raised above its present level, the water soon filters off in other directions.

It is estimated that the average daily supply of water in the underground reservoir amounts to about 332,000 gallons, but, owing to leakage, only 133,000 gallons reach the town, and a further loss is occasioned by defects in the local distribution, for which earthenware mains are employed, the smaller ones being connected with the larger by tin pipes let into holes roughly bored into the earthenware.

A considerable diminution in the supply has been observed during the last few years, and this has been generally attributed to the proximity of the recently constructed Cadiz waterworks, but the Author differs from this opinion. He recommends, first, that the underground reservoir should be thoroughly cleansed, repaired, and deepened; secondly, that the water should from thence be pumped into a reservoir placed considerably higher up the hill, so as to increase the difference of level between the source and Puerto de Santa Maria; thirdly, that iron pipes carrying the water to a higher level in the town should be substituted for the covered stone channel and also for the earthenware distributing pipes;

and, fourthly, that a large distribution reservoir capable of holding from 60,000 to 70,000 gallons of water should replace the small vault at the entrance to the town. The cost of these works is estimated at about £42,000.

O. C. D. R.

On the Corrosion of Zinc by Water and Saline Solutions.

By H. Snyders.

(Berg- und hüttenmännische Zeitung, vol. xxvii., p. 212.)

The Author, as the result of a great number of experiments, announces the following conclusions:—

1. Zinc decomposes both dilute and concentrated saline solutions, without the intervention of free oxygen; water being decomposed with the formation of zinc oxide and liberation of hydrogen.

2. The above action is accelerated by the solubility of the zinc oxide in the saline solution.

3. Zinc oxide is soluble in 1 per cent. and even weaker saline solutions, the amount dissolved varying with different salts, being highest with those of ammonia. This appears to involve the production of free alkali, which forms a double zinc salt soluble under particular conditions of concentration and temperature. Both the hydrated oxide and carbonate of zinc are insoluble in carbonates.

4. When the saline solution is saturated with zinc oxide, the corrosion still proceeds, but the further quantity of oxide formed remains undissolved. The exact reactions taking place at this stage have not, however, as yet been completely investigated.

5. Zinc is more rapidly attacked when the solution contains oxygen free from carbonic acid, which has a direct oxidising action. In this case the saline solution only serves to renew the surface of the metal by dissolving away the oxide as it forms.

6. On the other hand, the action of carbonic acid as contained in air has a slightly retarding tendency, from the formation of a film of basic carbonate on the surface of the metal.

7. The corrosive and solvent action is greatest with solutions of chlorides and sulphate of potassium, and less so with alkaline and barium nitrates and sulphate of magnesium.

8. Solution of alkaline carbonates and sodium phosphate are without action on zinc when air is excluded; and even with excess of oxygen, 1 per cent. solutions have but slight effect, the metal being protected by the carbonate or phosphate produced. In more dilute solutions, however, some oxide is invariably dissolved.

9. In all cases the solvent action increases with increase of temperature; at the freezing-point of water it is extremely small.

10. Solutions of ammoniacal salt take up more zinc than those of the corresponding salts of the fixed alkalies. The metal keeps a bright surface, and no separation of oxide takes place in the solution, even when air has free access.

11. Hard well waters are without action on zinc, even when containing a considerable proportion of chlorides and sulphates. Soft waters, on the other hand, show a considerable solvent power, which increases in proportion as the sulphates and chlorides exceed carbonates and phosphates held in solution.

H. B.

The Ar-Men Lighthouse.

(Notices sur les Modèles, Cartes et Dessins relatifs aux Travaux des Ponts et Chaussées, Exposition Universelle à Paris, 1878, p. 247.)

This lighthouse is being built on a rock called Ar-Men, situated near the extremity of a reef, extending in a westerly direction from the Isle of Sein, department of Finistère, across which there is a very strong run of tide. Two lighthouses were erected many years ago, one on Raz Point, and the other on the island, to warn vessels when approaching the reef. It was found, however, that the lights were too far from the extremity of the reef to form a thoroughly efficient warning, and, except in clear weather, could not be seen at a sufficient distance; and in 1866 the erection of a new lighthouse was decided on. The rock consists of hard gneiss, only 5 feet above water at the lowest tides, and the sea is seldom calm in that locality, as there is no shelter from westerly winds, or from those between S. and E.S.E. The run of tide is also very strong, attaining occasionally a velocity of over 8 knots an hour. The work was commenced in 1867 by boring holes in the rock 1 foot deep and about $3\frac{1}{4}$ feet apart, for receiving iron dowels. The holes were bored by the fishermen of the island, who, furnished with life-belts, seized every possible opportunity of landing on the rock. At the close of the first season seven landings had been made, and eight hours of work done, resulting in the boring of fifteen holes. In the following year forty additional holes were bored, and the rock partially levelled for receiving the masonry. In 1869 some of the dowels, $2\frac{3}{8}$ inches square and $3\frac{1}{4}$ feet long, were cemented in the holes, and 33 cubic yards of masonry were built, being set in neat Parker-Medina cement. By the close of 1877 the lighthouse had been raised $40\frac{1}{3}$ feet above the highest tides, the amount of masonry laid being 920 cubic yards, and it is anticipated that the lighthouse will be completed in three years. Since 1871 Portland cement has been exclusively used, and the lower portions of the structure are to be pointed and possibly coated over with it. The light is to be a flashing one of the second order, the base obtainable being considered too small

for a light of the first order; it is to be raised 94½ feet above the highest tides. From the base to high-water level the diameter of the lighthouse is 23 feet 7½ inches, built of solid masonry, and 22 feet 7¾ inches for the next 10 feet in height. There will be seven floors in the building, one of which will be set apart for the fog signal. The thickness of the walls diminishes from 5 feet 7 inches at the basement to 2 feet 7½ inches below the top of the cornice. A table gives various details of the progress and cost of the work from its commencement to the end of 1877, at which time 517,136 francs (£20,685) had been expended for a height of 16·7 mètres (54·77 feet).

L. V. H.

Formulæ for Predicting the Working Cost of any Railway.

(Giornale del Genio civile, an. xvi., p. 367.)

The offer of a premium by the Hungarian Academy of Science induced Herr Julius von Szabo, Professor of the Imperial and Royal Polytechnic School at Buda-Pesth, to inquire whether it is possible to calculate beforehand, with some exactitude, the proper working expenses of a railway; which must, necessarily, vary according to the system on which it is constructed and worked, and according to the traffic.

As the result of his studies he has compiled a formula, by means of which he considers it to be possible to calculate the cost in any particular case, and to solve many problems connected with the expense of working. Among these are included the reform of the tariff, the question of secondary railways, the determination of the most convenient gradients, and the like.

The subject is pursued by the investigator into minute detail. In very summary abstract, the Paper is divided into the four heads of (1), Nature and division of working costs; (2), determination of constants; (3), annotations on the formulæ; and (4), application of the formulæ.

Under the first head, Herr von Szabo considers that the working costs of every railway are divisible into the four groups of (1), General expenses of administration; (2), cost of watching and maintenance of way; (3), cost of the commercial service of the traffic; and (4), cost of traction and workshops. The first of these heads includes 2, the second 7, the third 5, and the fourth 7, subdivisions. These different elements of cost are severally affected by (1), the local conditions of the line; (2), the annual transport, per kilomètre, of passengers and goods; (3), the number and weight of the trains, reduced to kilometric tons gross; and (4), the gradients of the line. By the detailed application of these elements of variation to the original factors the detail of the

ultimate formula, $K = k_1 + k_2 + k_3 + k_4 + k_5 + k_6$, is rendered so complex that one example only can be cited to give an idea of the method. It is as follows:

$$k_6 = f s (r + \phi) \left[\frac{T'}{t'} (a' t' + S') + \frac{T''}{t''} (a'' t'' + S'') \right].$$

Here k_6 represents the group of locomotive costs, which are regarded as depending partly on gross mile-tonnage, and partly on gradients. The small letters are various co-efficients; ϕ denotes the mean gradient; T' is the unit of passenger traffic; T'' that of goods traffic; S' is the weight of the passenger engine; S'' that of the goods engine; r denotes the friction and the resistance of the train; s the weight of fuel, and f its cost per ton.

For the determination of the value of the constants, Herr von Szabo has made use of the detailed statistics of eleven German railways. For arriving at the values of T' and T'', it is estimated that each passenger, with the baggage not charged for extra, weighs 110 lbs. For the dead weight it is assumed that each pair of wheels carries, in passenger vehicles 6·617, and in goods vehicles 5·510 lbs. The locomotives weigh 40 tons each for passenger, and 45 tons for goods, trains. But the very first element of the general equation, viz., k_1, is said to vary from 260 to 2,600 francs, so that the average of 1,130 francs must be extremely rough.

The value of the formulæ thus arrived at has been tested, as stated by Herr von Szabo, by their application to the accounts of four inclines of very steep gradient, to those of 90 kilomètres of Norwegian railways, and to those of narrow-gauge railways in Switzerland. The ultimate sums arrived at by formula and by actual analyses of accounts are very close, but the several items vary by sometimes as much as 10 per cent. in either direction. It may be of use to place on record some of the actual facts collected by Herr von Szabo for the basis of his calculations.

PARTICULARS of LINES of STEEP GRADIENTS in AUSTRIA.

—	Semmering.	Poretta.	Giovi.	Sudbahn.
Length of line . . kilomètres	41·12	39·74	10·48	18·87
Sums of ascents . . . mètres	678	838	272	..
Train kilomètres run	374,672	218,319	152,063	8,897,826
Mean weight of train . . tons	125	115	90	190
" " engine . . . . "	57 to 67	53	60	..
	Francs.	Francs.	Francs.	Francs.
Cost of traction per kilomètre run	0·29	0·34	0·27	0·16
" grease . "	0·045	0·045	0·085	0·03
" repairs . "	0·223	0·365	0·50	0·16
Maintenance of way per kilomètre	5,085	1,858	5,607	..
Watching " "	807	1,187	2,918	..
Traffic charges	505	1,575	..	..
Direction of special service . .	480	955	1,090	..

Applying Herr von Szabo's notation to the above data the following result is obtained:—

——	Semmering.	Poretta.	Giovi.	Sudbahn.
Number of passenger trains $\frac{T'}{t'}$ . . .	730	730	1,460	1,227
" goods " $\frac{T''}{t''}$ . . .	838 ?	4,764	13,050	3,488
Gross weight of passenger trains $a't'$. tons	73,000	73,000	146,000	122,700
" " goods " $a''t''$. "	931,000	558,800	1,599,000	773,150
Total weight $= \frac{T'}{t'}(a't' + S') + \frac{T''}{t''}(a''t'' + S'')$. . . . . "	1,607,050	923,000	2,176,500	1,131,600
Part of k_3 deducible from above . francs	6,582	5,248	16,832	2,828
Total value of k_4 adding the constant 3,285 . . . . . "	9,867	8,533	20,117	6,113
Total value of k_3 . . . . "	6,875	5,578	9,615	..
Mean gradient $= \phi$	0·008	0·011	0·013	..
Maximum gradient	0·025	0·025	0·035	..

The two main conclusions which Herr von Szabo derived from his analysis of the accounts of so many lines, and which are independent of the value of his formulæ, are these:

1. The cost of working is proportionate to the first power of the gradient ϕ.

2. The speed of the trains has no perceptible influence on the cost of maintenance of way.

Tables are given of the wear of rails on gradients varying from 1 in 150 to 1 in 1,000, in which the result of a long series of observations made in 1872 by Herr Rockert is compared with the outcome of Herr von Szabo's formulæ. The co-efficient of wear rises from 0·280 for a gradient of 1 in 1,000 to 1·870 for a gradient of 1 in 150, according to the first; while, according to the formulæ, they are respectively 0·28 and 0·187. (The English expression, 1 in 1000, is represented in Herr von Szabo's notation as $\phi = 0·001$, and 1 in 150 as $\phi = 0·000666$.) The Author thus claims an exact accord between theory and practice in this item. He considers that the influence of the horizontal blows given by the wheels to the rails is too inconsiderable to be taken into account. He concludes by the remark, that if the cost of construction increases or diminishes according to the nature of the gradient, the solution of the general problem will be attained by making p, the cost of the line, a fraction of ϕ, the product. When this cannot be done, the problem can only be empirically solved.

F. R. C.

Elements of the Cost of Transportation.

(Appendix to the Thirtieth Annual Report of the Richmond and Danville Railroad Company. Richmond (U.S.), 1878.)

The costs of railway transportation are divided, according to the Author of this report, into the two classes of fixed and variable expenses. The former, however, are only spoken of as "fixed" with reference to any particular road. The latter includes items of expense which, whether singly regarded or grouped together, vary in widely differing ratios, even on the same road. These two main branches of outlay have been arranged by the General Superintendent of the Richmond and Danville railroad under the following descriptions:—

1. Fixed expenses. These include the cost of maintenance of works, repairs, and removal of all perishable structures, and general expenses of administration. The conditions which affect the item are the volume of business, and the cost of maintaining the roads and structures.

2. Terminal or station expenses. These include the cost of ticket distribution, porterage, loading and unloading goods, warehousing, and the items usually brought under the head of traffic charges.

3. Train mileage expenses, including repairs and supplies of locomotives, switchmen, signals, train guards, and the like.

4. Car mileage expenses, including repairs of vehicles, grease, waste, &c.

5. Tonnage expenses. These consist in fuel for locomotives, and maintenance and renewal of way.

To these must be added (6) interest on cost of road; and (7) interest on plant or equipment, including a proper allowance for depreciation.

As the fixed expenses are mainly affected by the volume of business and the nature of the country traversed, the terminal expenses are rated in an inverse ratio to the length of travel, and are affected by the nature of the commodities as to travelling. The train mileage expenses are affected by gradients, by curves, and by speed. The car mileage expenses depend on the relative bulk of the objects transported, and on the direction or balance of the traffic. The cost of fuel and of permanent way materials affects the tonnage or locomotive expenses. The cost of the line affects the interest on road; and the regularity of the traffic affects that on the equipment of the lines.

NOTE.—On a longer line the terminal expenses will be proportionately diminished. But it is not obvious why no allowance is made for terminal expenses for a return train.—F. R. C.

A formula for the general cost per ton per mile of freight transportation (omitting items 6 and 7) is constructed as follows:—

Let F = fixed expenses per train mile;
S = terminal expenses per ton of freight;
T = train expenses per train mile;
C = car expenses per car mile;
G = tonnage expenses per ton mile goods;
l = load per car in tons;
h = length of haul;
n = number of cars per train;
W = weight of engine;
w = weight of cars.

Then—

$$\text{Cost} = \frac{S}{h} + \frac{F + T + Cn + G\,(W + (w + l)\,n)}{l.n.} \quad (1).$$

When the traffic is sufficient to give full loads for trains between two points, this formula is applicable. For average trains on the lines of the Richmond and Danville Railroad Company, the following values, which are substituted for the above symbols, have been tabulated. For a distance of ten miles—

$$(1) \quad 2\cdot087 + \frac{33\cdot20 + 24\cdot28 + 10\cdot69 + 22\cdot54 + 7\cdot79}{61\cdot51} = 3\cdot600 \text{ cents.}$$

If the train return empty, an additional cost per mile is necessary—

$$(2) \quad \frac{33\cdot20 + 24\cdot28 + 10\cdot69 + 22\cdot54}{61\cdot51} = 1\cdot475 \text{ cent,}$$

making the cost per ton of freight per mile 5·163 cents, or 2·58*d*.

For the transportation of passenger trains, the same symbols as before are used, with the exception and addition of the following:—

C = car expenses per foot run of car (in train) per mile;
G = tonnage expenses per mile-ton of cars and loads;
R = space occupied in length of car;
w = weight of cars per foot;
l = load of cars per foot.

The formula given for the cost per foot of car per mile is:—

$$(3) \qquad \text{Cost} = \frac{S + F + T}{R} + l + G\,(w + l).$$

The value of the above, according to the Author, on the lines analysed, is as follows:—

$$(3) \quad \frac{6\cdot209 + 22\cdot037 + 10\cdot995}{151\cdot53} + 0\cdot0396 + 0\cdot2367\,(0\cdot398 + 0\cdot0833) = 0\cdot4 \text{ cent per mile.}$$

The trains afford seats in the ratio of one passenger per foot of train. But it is rarely the case that one seat in four is occupied. The actual average on the Richmond and Danville railroad for three years was 26·5 passengers per train, or 5·7 feet per passenger, making the cost per passenger mile 2·26 cents, or 1·13*d.*

The train resistances are described as due to (1) friction, (2) grades, (3) curvature, and (4) speed. The first is stated at about 0·3 per cent. or 6 lbs. per ton of 2,000 lbs. The third is estimated at ½ lb. per ton for every degree of deflection for chords of 100 feet. The fourth is estimated from analysis at about 7·3 per cent. increase in expense for an increase of 33·3 per cent. in speed. The cost of mail matter and express freight is also entered into in detail. Tables are annexed showing the details of cost, and the mode in which they are divided; the elements of the cost of passenger trains for three years; the elements of the cost of mixed trains; the increase of cost from a level to a gradient of 1 in 100, for engines of 8,000 lbs., and of 16,000 lbs., traction power; the mileage of cars; the mile-feet of passenger cars; the mile-tons of cars and loads; and the gross mile-tonnage of passenger and goods traffic on the Richmond and Danville and North Carolina railroads for the three years 1875, 1876, and 1877.

F. R. C.

NOTE.—The use of a foot in length of train as a unit is novel, and has some advantages, especially for calculations of atmospheric resistance at increased speed. From the details of trains experimented on by the Royal Commission on Railway Accidents (Report for 1877, p. 97) it may be calculated that on the London and North-Western railway it takes 1·3 foot run per seat, the train weighing 6 cwt. per foot run. On the Great Northern the allowance is 1·25 foot per seat, 5·72 cwt. per foot run. On the Lancashire and Yorkshire there is 1·14 foot of train per seat, and a foot of train weighs 5·37 cwt. On the Caledonian there is a seat for every 1·03 foot of train, and a foot run weighs 5·05 cwt. On two experimental Midland trains the accommodation was respectively one seat per 1·16 foot, and 1 seat per 1·04 foot run. The Midland Company have not furnished the weight of their carriages.

It may be added that the tare of the freight on the American lines in question averages 65 per cent. of gross load—which is almost exactly the percentage on the railways of New South Wales, according to the elaborate report of Mr. Rae, the Government director. The passenger tare is as much as 97 per cent. of the gross load, which is unusually high. This is owing to the small proportion of seats actually occupied on the average.—F. R. C.

Railway joining the Stations of Guillemins and Vivegnis at Liége.

By M. DEBEIL.

(Annales des Travaux Publics de Belgique, vol. xxxv., p. 323.)

This junction railway is 2 miles 770 yards long, of which a length of 1 mile 715 yards was designed to be in tunnel, and the rest in open cutting. The cost was estimated at £244,000. The strata traversed by the line consist of clay, schist, and the sand-

stone of the coal measures. Considerable difficulties were experienced in the construction of the line at several points. At one place the clay, traversed by layers of gravel and sand containing water, began to slide, during rainy weather, and brought forward the retaining wall at the side of the cutting, and though the walls were strutted with timbers 13½ inches square, the motion was not completely arrested till an invert was constructed between the walls. In making a portion of the tunnel under St. Gilles the ground became very soft, and oozed through the sheeting at the sides, forming hollows behind and producing slips which threatened injury to the Liége and Brussels railway, and damaged the roof of the tunnel. The water-bearing stratum being above the foundations of the tunnel, two rows of sheeting were driven into the solid ground along the line of the sides of the tunnel, with cross rows of sheeting at intervals of 13 feet, and a series of masonry wells were sunk for pumping out the water. The earthwork was removed in open cutting; the invert of the tunnel was built first, then the abutments were constructed, and the wells, being filled with concrete as the work proceeded, were incorporated into the abutments and haunches of the arch which was built last. This method was adopted for a length of 88 yards, and the cost of this portion of the tunnel was £106 per lineal yard. In another part made ground was unexpectedly met with, and a tunnel was substituted for an open cutting. The excavation for the side walls of this tunnel on one side, where the ground was solid, was performed by means of a heading, but on the other side wooden shafts had to be sunk through the loose material. The side walls were built first; the arch was then turned in lengths of about 15 feet, and lastly the invert was put in. This tunnel cost £66¾ per lineal yard. The tunnel under Mount St. Martin, which traverses clay and schist intersected by bands of psammite and sandstone, was partly built in open cutting and partly tunnelled under houses, some of which were inhabited during the progress of the work. Along St. Séverin Street, where the tunnel came close under the foundations of the houses, the buildings were bought by the State and propped up before the work under them was commenced. Wherever the foundations were laid bare in the excavation for the tunnel they were underpinned with wooden frames, and on the completion of the arch wooden wedges were driven in above it to diminish the settlement. The cracks produced in the houses were repaired, and the houses relet. The great sewer of St. Séverin had to be diverted. The cost of the tunnel was nearly £50 per lineal yard. A settlement of the church of St. Séverin occurred during the construction of the retaining walls at Volière Street. In the tunnel under Pierreuse the excavations of an old sandstone quarry were met with, and the propping up of the roof of the quarry and the carrying the tunnel through the rubbish and insecure cavities added considerably to the difficulties of the work, and the tunnel had to be lengthened. The addition of an invert to all the tunnels, the increase in

thickness of the masonry, the substitution of tunnel for open cutting in two instances, and other extra expenses brought up the actual total cost to £338,460, the works alone costing nearly £41 per lineal yard. The details of the works are fully illustrated in the series of plates accompanying the article.

L. V. H.

On the Working of the Southern Railway of Austria in 1876 *and* 1877. By A. Gottschalk.

(Mémoires de la Société des Ingénieurs civils, 1878, p. 440.)

This is an amplification of a previous Paper by the same Author published in 1877, which described the results of the working of the same railways from 1872 to 1875.[1] The network comprises 354 miles of main line from Vienna over the Semmering Pass to Trieste, and 688 miles of branches from the main line to Laxemberg, Ofen, Uj-Szöny, Œdenburg and Kanizsa, Villach, Sissek, Carlstadt, Cormons on the Italian frontier, Vordernberg and Bares; besides 191 miles from Kufstein to Ala over the Brenner summit, called the Tyrol railway; 130½ miles from Villach to Franzenfeste, called the Pusterthal railway, and 34½ miles from Saint-Peter to Fiume on the Adriatic, a total extension of 1,398 miles. A general plan and longitudinal sections of the different lines accompany the Paper, and elaborate Tables are annexed, in which the cost of traction and maintenance, the amount of traffic in passengers and goods, the consumption of fuel, and the general results on the several lines from 1868 to 1877, during which period the railways have been under the management of the Author, are carefully set forth and year by year compared. Two state lines in Istria, from Divazza to Pola and from Confanaro to Rovigno, together 90 miles in length, have been also administered by the company since 1876, but the results of their working are not shown.

It is stated that the two principal subjects to which the attention of the company has been directed since the date of the previous report, have been: 1st, the introduction of all the improvements in engines and rolling-stock which were prescribed by the new rules at Constance in June, 1876, by the delegates from the German and Austro-Hungarian railways at the general assembly of the German Railway Union; and 2ndly, the study and introduction of the most improved system of continuous brake.[2]

The marked diminution in the cost of working these railways, which had previously been observed, has continued down to the most recent date, and this is partly due to the lower cost of fuel, &c., and partly to the constantly increasing power of the engines employed. The substitution of engines with eight coupled wheels

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xlix., p. 315.
[2] *Post*, p. 316.

and separate tenders for the tank engines with six coupled wheels formerly used, has on the Karst incline alone (between Leybach and Trieste) occasioned an economy of nearly £40,000 between the years 1872 and 1877. By reducing the number of goods trains, it has permitted the company to lessen the number of drivers and stokers by more than one-third, and to greatly diminish the consumption of fuel. The average train-weight on this section has been increased from 177 tons 7 cwt. in 1867 to 220 tons 18 cwt. in 1877, or at the rate of 24½ per cent., with a reduction of 8 per cent. in the average train-cost per mile, or of 20½ per cent. in the cost per ton of goods, and this on the total amount of goods carried in 1877 is equivalent to a saving of no less than £90,132.

The general expenses during the same period have diminished 23½ per cent., and although on the other hand the expenses included under the head of renewals and repairs of rolling-stock have increased 23 per cent., the total average cost per train-mile in 1877 was 51·6 per cent. less than in 1860, notwithstanding the greatly increased weight of the trains.

A summary of the results obtained in 1876 and 1877 on the Semmering incline is given in the Tables Nos. 3, 4, 5, and 6, and shows that they compare very favourably with the results of previous years. Compared with the year 1867, the cost of working had diminished in 1876 by 18·4 per cent., and in 1877 by 22·5 per cent. per train-mile. It has been proved by ten years' working over the incline that, notwithstanding that some curves have a radius of only 600 feet, the engines can be placed without the slightest inconvenience at the head or at the tail of the trains indifferently.

The diminution on the Tyrol line in 1877, as compared with 1868, in the average train-cost per mile was 21·4 per cent., and on the cost per ton per mile 48·5 per cent. The average weight of the trains on this section has increased as much as 53 per cent., viz., from 112 tons 8 cwt. in 1868 to 170 tons 10 cwt. in 1876, and to 172 tons in 1877.

Supposing that the same traffic had existed in 1868 as in 1877, this general reduction in the working expenses would on the united lines exceed the sum of £180,000, which is about 40 per cent. of the total working expenses in 1877 on the South Austrian railways.

O. C. D. R.

Continuous Brakes on the Southern Railway of Austria.

By A. Gottschalk.

(Organ für die Fortschritte des Eisenbahnwesens, 1878, p. 184.)

The subject of continuous brakes, as the Author observes, is of the utmost importance on this railway, inasmuch as it includes the two formidable passes of the Semmering and the Brenner. The

ordinary hand brakes were some time back replaced (first on the Brenner only, and afterwards on other portions of the line) by the Le Chatelier counter-pressure brake, which, by means of a special pipe, reverses the action of the steam in the cylinder, and applies the whole power of the engine to arrest the train. These brakes did excellent service for the mountain inclines, and for slow goods trains, but were not sufficiently prompt in application for quick passenger traffic. Experiments were made in 1873 with the electric brake of Chapin, and the progress of the question in England and America was closely watched. The Westinghouse brake was rejected as being too complicated, and not adapted for a line where the gradients were so variable that the brake was continually being put on and off; and the Smith vacuum brake, which the Author had seen tried on the Northern railway of France, was preferred, and first started on the Semmering in November 1876.

The results were very satisfactory, especially as regards the easy and rapid manipulation of the brake. A simple arrangement was devised by which the admission of steam to the ejector, and consequent application of the brake, could be graduated with the utmost nicety. Subsequently, Mr. Hardy, manager of the railways workshops at Vienna, effected an important improvement by substituting for the accordion-like india-rubber bag of the Smith brake a hollow drum of cast iron, with a head of the same metal, but somewhat smaller in diameter, and connected to it all round by a leather diaphragm. When the vacuum is formed in the interior, this head is driven inwards, and by its motion applies the brakes. This design (which, unknown to Mr. Hardy, had been anticipated by Dutremblay and Martin in 1860) is much more durable, and at the same time gives a much smaller vacuum space, thereby increasing the rapidity with which the brake is applied. The good opinion which the Author was led to form of the Smith vacuum brake, as thus improved, was confirmed by a visit which he paid to England, in the course of which he inspected the chief systems of brakes there in use. The grounds of his preference are cheapness and simplicity of construction, its adaptation to the requirements of daily use, and its special suitability to long and variable gradients. He recommends that two separate pipes should be connected with the ejector, one leading to the engine tender, the other to the carriages, so that the brakes on the former would be rapidly applied in any case, even when the latter was partly neutralised by the breaking of a coupling. Smith-Hardy brakes are now applied to eighteen engines and fifty carriages on this railway.

With regard to the material of the brake-blocks, long experience has shown that cast iron (from charcoal-iron pig) is to be preferred to wood, which is apt to skid the wheels; to wrought iron, which, from its inequalities, is apt to injure the tires; and even to cast steel, which has been extensively used in Germany.

The application of the automatic principle to brakes, so that

they would put themselves on when required, as by the breaking of a coupling, would be a great advantage; but with any of the arrangements at present devised, it seems to be bought too dear, on account of the excessive complication which it necessitates. Experience shows that with passenger trains such accidents are very rare; and for goods trains, where they are much more frequent, continuous brakes cannot as yet be thought of. The Author, however, calls attention to an excellent system of automatic brake, worked, like the Heberlein brake, by the momentum of the train itself, and designed by Herr Becker, chief inspector of the Emperor Ferdinand Northern railway.

W. R. B.

Railways in Asiatic Turkey. By F. Rolin.

(Mémoires de la Société des Ingénieurs civils, 1878, p. 309.)

The Author gives a comprehensive description of the various lines of railway projected from the Mediterranean, the Black Sea, and the Sea of Marmora to Bagdad and the Persian Gulf. The topography and resources of the country, its population and existing means of transport, its adaptability for railway construction, and the nature of the local traffic on the several routes, are carefully described, and the proposed lines are shown on a map.

The topography of Asia Minor is described by the Author in four principal divisions. Of these the first is the most western portion, and includes also the lines of the coast; it is characterised by innumerable rivers and fertile valleys, and is one of the richest provinces of the Ottoman empire. Beyond these is the second, an inverted triangle, of which a line from Broussa to Trebizonde may be regarded as the base and the town of Karaman the apex. This part of the country is one vast elevated plateau, averaging from 2,000 to 3,300 feet above the sea. Further to the east is the third, comprising Armenia and Kurdistan, which are mostly mountainous with peaks rising to a great height. Erzeroom, in the centre of this district, is 6,150 feet, Kars 6,280 feet, and the Ala Dagh 10,000 feet above the level of the sea. The fourth division is that of the Taurus and Anti-Taurus, chains of mountains which run from east to west and separate Asia Minor from Syria and Mesopotamia (called by the Turks Irak).

The proximity of the elevated plateau to the coast opposite Constantinople will occasion serious difficulties in the construction of a railway starting from the Bosphorus or the Black Sea, because it would be necessary, after leaving Ismid, to employ gradients of 1 in 50, in order to reach the summit. Starting on the other hand from Smyrna on the western coast, from whence a branch line is proposed, the distance is greater, and the rise much easier.

Again, south of the Taurus mountains and of Kurdistan, the Tigris and the Euphrates water an immense basin which is called

Djeul, or desert, and is separated from Syria and Palestine by chains of mountains running from north to south. The great basin of Mesopotamia, which is filled with ruins denoting former wealth and splendour, lies more on the direct route between Constantinople and Bagdad, but, excepting in the immediate neighbourhood of the towns, Mesopotamia is now little better than a desert and a swamp. The ancient canals are filled with silt, and the water-dams in the rivers, which formerly served for irrigating extensive districts, have been mostly destroyed. Thus, the more direct railway across the Mesopotamian plains becomes impracticable, and has caused the route through the southern part of Kurdistan and along the left bank of the river Tigris to be preferred.

Before entering upon a detailed description of the projected railways, the writer offers some observations on the state of the existing means of transport in Asiatic Turkey. There are hardly any roads, excepting those constructed by the ancients of large blocks of stone. These are now in a state of ruin and are available only at certain points, as at the passage of swamps, or over hills. At other places the roads are mere tracks, and travellers strike into the fields to the right or left, according to the state of the ground. Bridges are extremely rare. This notwithstanding, there is a large traffic on some of the roads. Broussa alone furnishes a traffic of 40,000 passengers and 12,000 tons of merchandise per annum; and at Ismid 100 tons a day pass along the road. A few modern roads have been constructed, *e.g.*, from Trebizonde to Erzeroom, Guemlik to Broussa, Broussa to Moudania, and Beyrout to Damascus, and they are kept in a fairly good state of repair. Water communication is extremely rare in Asiatic Turkey, and is limited to a service of steamers between Bagdad and the Persian Gulf, to a few rafts which navigate the waters of the Upper Tigris as far as Mossoul (Nineveh), and to floating timber down some other rivers. Horses are used only for the carriage of light goods and for the conveyance of the post, which occupies twenty-two days between Ismid and Bagdad.

The camel is more generally employed, and is an economical beast of burden. It has been proved that from Smyrna to Allaheheir camels convey goods more economically, although much slower than the railway which runs between those two cities. In Anatolia the buffalo is preferred and is used for draught.

Owing to the absence of roads the produce of the country cannot be conveyed to any distant markets. In 1872 there was a disastrous famine in Anatolia, and many villages were depopulated, whilst in Kurdistan the harvest was so abundant that the corn rotted where it grew, and this circumstance had much to do with the determination then arrived at by the Turkish Government to order a complete survey to be made of the lines which would be necessary in order to supply the whole of Asia Minor, Mesopotamia, and Syria with railway communication. This survey was proceeded with by Mr. Wilhelm Pressel, in 1873 and 1874, but in the following year it was interrupted by the events which occurred

in the Balkan peninsula. In Asia Minor there are now 257 miles of railway open to traffic, viz., from Haidar-Pacha to Ismid, 57; from Smyrna to Aïdin, 81; and from Smyrna to Allaheheir, 118 miles. There are also 81 miles in course of construction, viz., from Moudania to Broussa, 26, and from Ismid to Melredjé 55 miles.

The projected lines are:—

1. From the Mediterranean to the Persian Gulf, starting from Tripoli and proceeding *viâ* Homs, Tadmor (Palmyra), Déir, and the Euphrates Valley.

2. From the Mediterranean to the Persian Gulf, starting from Suedia, Alexandretta or Tripoli, and proceeding by Aleppo, Orfa, Mardin, and thence to Tischabour and Moussoul, Kerkouk and Bagdad, following the left bank of the river Tigris.

3. From Constantinople to Mardin, and thence by route No. 2 to Bagdad, with numerous important branches.

1. If the railway started from Tripoli, it would be necessary to follow the coast northwards for about 22 miles, as far as the mouth of the Nahr-el-Kébir river, and then to follow the valley formed by it up to the plateau which separates it from the Orontes river, the course of which would be followed down to Homs, which is about 1,500 feet above the sea. This portion of the line offers no serious difficulties. An undulating plain, presenting still fewer engineering difficulties, extends from Homs to Tadmor (Palmyra), and from thence onwards to Déir on the Euphrates. The ground rises to an elevation of 2,000 feet at Tadmor, and falls again to 670 feet above the sea level at Déir, but it is a desert throughout the whole distance, with the exception of the environs of Palmyra. The gradients on the route would be light, but the materials for the construction of the railway would be altogether wanting. There is no timber, and the almost total absence of population would make the establishment of a railway quite impracticable. From Déir the valley of the Euphrates would be followed to Bagdad. The river is not navigable. Possibly the construction of a railway might eventually lead to the irrigation and cultivation of the plains of Mesopotamia, but there is one fatal objection to this line, viz., that the country is everywhere inhabited by wandering tribes who would interfere with the security of the traffic, and in the opinion of the Author this route ought certainly to be rejected.

2. The Author recommends the Port of Suedia in preference to either Alexandretta or Tripoli, as the starting point for the railway. Alexandretta, although the best port on the coast, is an open roadstead in the Gulf of Scandaroon; towards the east the bay is protected by high mountains, but when the wind blows from the N. or from S.E.E. the anchorage is dangerous. The town of Alexandretta is built on ground which is scarcely 3 feet above the level of the sea, and in stormy weather the streets are inundated. Behind the town are marshes only, which are a few inches above the level of the sea. It is much subject to earthquakes. The

distance in a straight line from Alexandretta to Aintab is 81 miles, but if a railway had to be constructed to this point, it would require to follow a direction which would lengthen the distance to 144 miles, and the difficulties of construction would be very great, involving gradients of 1 in 40, and a tunnel at Beilan under the Karadagh summit upwards of 6 miles in length, besides four other shorter tunnels, and a great number of bridges and viaducts, heavy cuttings and expensive alterations of the river Afrin. There is no existing road to facilitate the work, a complete absence of timber, and the population very sparse.

Tripoli is, next to Alexandretta, the best port on the coast, and a line constructed *viâ* Homs and Aleppo to Orfa would offer no engineering difficulties, but the distance from Tripoli to Aleppo would exceed that from Suedia by 120 miles; that alone is sufficient reason for preferring the latter as a railway terminus.

Suedia, a few miles south of Alexandretta, is the ancient port of Seleucia. It lies between two ranges of mountains, called Djebel-Mouza and Djebel-Ahra, at the mouth of the river Orontes. The depth of water at 220 yards from the shore is 13 feet, but at 550 yards there are 33 feet of water with a solid rock bottom. The port is entirely protected by the Djebel-Ahra mountain from the south wind, so dangerous in the Mediterranean, and the Ras-el-Chanzir, which separates the bay from the Gulf of Alexandretta, protects it from winds from the north. The winds from the land side do not affect it, and the harbour might easily be protected on the west side by the construction of a breakwater.

The railway would follow the banks of the Orontes with a rather steep rising gradient as far as the plain of El-Mak, and thence to Antioch (14½ miles), and Aleppo (86 miles), over ground which, between those two cities, is mostly at an elevation of about 1,150 feet above the sea. Its construction presents no serious difficulties. From Aleppo to Biredjik on the Euphrates, the average level of the ground does not vary much, but it is intersected by a great many ravines. Throughout this whole district (172 miles) the country is thickly populated. Between Biredjik and Orfa (250 miles from Suedia), the line crosses the Euphrates and some high ground beyond it. After passing Orfa, the direction would be determined by the point at which it were considered advisable to join the main line from Constantinople to Bagdad. This might be at Diarbekir, but the Author would prefer the shorter line to Mardin which lies further south. Between Orfa and Mardin, a distance of 120 miles, the railway would cross the great Mesopotamian plateau, called the Desert, which is nearly level throughout, and would require scarcely any earthworks. Although called the Desert, the land is all well watered and extremely fertile, and is only not cultivated on account of its having until recently been occupied by wandering Bedouins, who were robbers but have now been completely subdued by the Government.

The Author strongly advises the adoption of this route, which he points out is the same as that recommended by Mr. Latham,

with the exception of the choice of Suedia for the terminus instead of Alexandretta, which was preferred by Mr. Latham. The expense of constructing a good port at Suedia would, he says, be very much less than the mere passage of the Karadagh at Beilan, but he acknowledges that other considerations, which could only be determined by a careful study, might possibly incline the balance in favour of Alexandretta.

Being so decidedly in favour of this route, the Author proceeds to describe with great minuteness the conditions of the country which would be traversed by it.

Syria, which the Arabs call Bahr-es-Cham (the "country to the left"), is intersected by four principal ranges of mountains, spurs of the great Taurus chain. The first of these forms the northern boundary of the province, and reaches its highest elevation at Beilan opposite to Alexandretta, between which and Suedia there runs another spur called the Djebel-Mouza. The latter continues south of the river Orontes as far as the Gulf of Tripoli, where it is interrupted by a broad valley leading up to Homs, on the opposite side of which the well-known chain of the Lebanon commences, running from that point in a south-easterly direction.

From Suedia the railway ascends the Orontes river rising 280 feet to Antioch (pop. 10,000), which stands on the broad plain of El-Mak; after passing this point the valley winds back again to within 3 miles of the coast, and, becoming very much more confined between rocks, will necessitate a tunnel at this point nearly 400 yards in length; it then again expands like a funnel, and 6 miles beyond Antioch a marshy plain is reached, also called El-Mak, so nearly level that throughout its whole extent of 15 miles the height does not vary so much as 3 feet; it is intersected by several streams, all however of insignificant dimensions. A little further on another marshy plain has to be crossed, called the Amk, which is almost entirely surrounded by mountains, and then some hilly ground, on the other side of which lies the table-land on which Aleppo is built (1,250 feet above the sea), which extends eastward as far as the Euphrates.

Aleppo is an important city of 110,000 inhabitants. After passing it the country is intersected by numerous valleys, and before reaching the Euphrates the line must cross another range of hills; but this will not be difficult, as a canal built by the ancients indicates the direction which will have to be followed. Should the railway be required to pass by Aintab (pop. 18.000), which is surrounded by mountains, it will have to come back as far as Jaghden in order to reach Biredjik on the banks of the Euphrates.

At the most favourable spot for crossing this river it is 275 yards in width when the waters are low; but in winter time this width is doubled, and in times of great floods it is sometimes as much as 800 yards.

The left bank is very precipitous, and the town of Biredjik (pop. 4,000) is built on terraces rising one above the other from the banks of the river to the summit; but by crossing at about

3 miles above the town, the course of a small stream will allow of the railway being carried without difficulty to the plateau beyond, and then to continue nearly on the level as far as Orfa (250 miles from Suedia), over ground which, however, is so much intersected by streams that the works will involve considerable expense. Orfa is a large town (pop. 35,000) built on the side of a mountain called the Nemrod Dagh, running from north to south, and this and another mountain which runs parallel to it will have to be crossed by the line. These mountains overlook an immense plain which stretches to the south-east as far as Bagdad, and over a part of which the railway will be carried to Mardin, the proposed point of junction with the line from Constantinople.

The total length of the railway from Suedia to Mardin is 370 miles, of which 209 are classed as easy of construction, 40 as moderately so, and 121 as difficult. It is stated that the total length of tunnels on this line would be about 2,750 yards, and that of the bridges exceeding 11 yards in length 2,000 feet.

The distance from Mardin to Diala, which is 10 miles beyond Bagdad, is 486 miles. Mardin is at an elevation of 1,950 feet and Bagdad at 165 feet above the sea level, and the railway makes the descent with considerable regularity. Its proposed course is through Tischaboor and Mossoul (the ancient Nineveh), both of which are on the banks of the Tigris, and Erbil, Altinkeupru and Kerkuk which lie a little to the east of it. Its course is in general over alluvial plains, with a scanty population, excepting in the neighbourhood of Mardin, Mossoul, Erbil, and Bagdad, where it averages from 60 to 90 per square mile. At Tischaboor the bridge over the Tigris would be of considerable magnitude, but there are no other works of much importance.

A table is given, showing the nature of the lands along the route, classed as cultivated, susceptible of cultivation, and waste; and they are in the proportion of 53 to 24 and 23.

The comparative difficulties of construction on these 486 miles, from Mardin to Bagdad, are also classed as easy on 268, moderately easy on 175, and difficult on 32 miles.

The entire length of the line which has been described, from the Mediterranean at Suedia to the terminus at Diala, below Bagdad, is 855 miles; and the Author estimates the cost of construction of the railway, if the ordinary gauge of 4 feet 8½ inches is adopted, at £10,880 per English mile, which price would include the rolling stock, and would make the total cost amount to £9,302,400; but a narrow gauge line might, he adds, be constructed for £4,924,800.

Between Suedia and Biredjik the native population will supply a sufficient number of workmen for the construction of the railway; but beyond that point the sparseness of the population will make it necessary to employ Arabs. The Arab labourers earn 6 or 8 piastres (*1s.* to *1s. 2d.*) a day; the Turcomans, who are less industrious, but more powerful men than the Arabs, earn 4 piastres (*7d.*), and their food. The Kurds have great physical strength

and are ambitious and persevering. Masons, carpenters, blacksmiths, and other mechanics are to be met with throughout the country. In general the country is poor in minerals, but iron ore is met with in the neighbourhood of Aintab. Timber is very rare, but other building materials are abundant throughout the whole route. The exports from the three seaports of Alexandretta, Beyrout, and Mersina amounted in 1871 to 630,735 tons.

3. Passing to the proposed railway from Constantinople to Bagdad the Author first examines the question as to which would be the preferable port to select as a starting point. Scutari, which is opposite to Constantinople, is, he remarks, well sheltered, but the water in the bay insufficiently deep for large ships to anchor there. He recommends that this port should be connected by railway with the much better port of Ismid in the Sea of Marmora, 60 miles to the east, which is built on the shores of a spacious gulf 20 miles across, where at five different points there is deep water and excellent anchorage for large ships. There is already a railway from Ismid (population 10,000) to Haidar-Pacha (57 miles), and an additional 2 miles would connect the latter point with Scutari. Ismid would then serve as the port for heavy goods, whilst Scutari might be preferred for passengers. The harbour of Ismid is about 6 miles inland; the railway southward would rise immediately with a steep ascent and gradients of 1 in 66 and 1 in 50 to the height of Goh-Dagh, following the banks of the river Kilessi; continuing the ascent from thence to the elevated main plateau previously described, and, after passing by Eski-Chéir (2,100 feet above the sea), would reach the fertile plain of Ada-Bazar, 30 miles from Ismid, upon which there are no less than 120 villages, and a population of 40,000, chiefly Greeks and Armenians, with a minority of Turks and Circassians. A river, called Sakaria, runs through this plain to the Gulf of Ismid, and although the works upon its banks will be very heavy, the defile which it creates is the best direction for the railway to take. Continuing the rise southwards, on emerging from the gorges at the head of this river, the left banks of which are of mountain limestone, and the right bank of granite, another great plain is met with similar in fertility to that of Ada-Bazar. The highest points of this plain are 3,600 feet above the sea, but it produces in great abundance wheat, cotton, and mulberry trees. Here, in order to reach the most elevated part of the plateau, the railway should leave the river Sakaria and follow one of its tributaries, the Kara-Sou. From Lefké (66 miles), through the gorge formed by this river, to Vezir-Hau, a distance of 12½ miles, the works of the railway will be very heavy, and require a succession of tunnels and viaducts; but from thence to Eski-Chéir (133 miles from Ismid), on the borders of the great plateau, the line is easy of construction, and runs through well-cultivated and populous valleys.

Up to this point the line is classed as easy of construction for about 40 miles, presenting moderate difficulties on about 80, and

great difficulties on 12½ miles. Building materials are abundant, with the exception of timber, ordinary labour plentiful and cheap, but skilled labour scarce, although Greeks and Armenians, who are very good masons and carpenters, would be met with. Judging from the existing traffic on the Ismid and Haidar-Pacha railway, the Author estimates the passenger traffic between Eski-Chéir and Ismid at 160,000 per mile per annum, the goods at 128,000 tons, and the gross receipts at £1,280 per mile per annum (say £24 10s. per mile per week), which he believes will be doubled in the course of a few years. The Author, after minutely describing the railway works between Ismid and Eski-Chéir, mentions three different routes from the last-mentioned town to Mardin, the point of junction with the Suedia and Bagdad line.

The first is across the plateau to Kutaya in the south, and from thence across the Taurus mountains to Aleppo, taking up at that point the route previously described from Suedia to Mardin; but this route is so difficult of construction, and of such little commercial use, that he does not advocate it.

The second is from Eski-Chéir to Baktasch, not far from Cesarea. This line crosses the great plateau to Sivri-Hissar, continuing thence north of the Lake Tous-Gueul, and along the Hélys river, until, at Baktasch, it suddenly diverges to the south, crossing the western extremity of the plateau of Tunuz to the towns of Nigdé and Allankichla, from whence it joins the first route.

The third line is that advocated by the Author, from Eski-Chéir to Cesarea. A project had been made running due east, and passing by Angora; but that would be so difficult and expensive, and of so little use for traffic, that it has been given up. From Eski-Chéir the line should run south-east across the plateau to Sivri-Hissar (200 miles from Ismid), and one of these two towns should be selected for the starting point of a branch to Smyrna, 331 miles in length, of which 120 are already constructed. Passing Angora (population 30,000), Ak-Serai, Newcher, and Nigdé, the railway continues upon the same plateau to the large city of Cesarea, with 60,000 inhabitants (456 miles), and thence bearing east by north arrives at Sirvas, from whence a branch is projected to the Port of Samsoom, on the Black Sea. At Sirvas, or Scharkisla, the main line bears again to the south-east, and passing by Kangal (from whence another branch line is proposed to Erzeroom, the chief town of Armenia), Malatia, and Djarbékir, reaches, at Mardin (875 miles), the point at which it would unite with the line from Suedia to Bagdad, previously described.

This line is the easiest and most economical to construct, and commercially it is also preferable, because, running parallel with the great river Sakaria, which traverses the centre of the country, it will be a great means of traffic from the coast to the most important towns in the interior of Asia Minor. It occupies also the most favourable position for securing the traffic between Constantinople and Persia and Georgia.

The Author gives the heights and distances from Ismid of the

following places touched by this railway, in order to give an idea of the general longitudinal section which it presents.

	Heights above the Sea Level.	Distances from Town to Town.
	Mètres.	Kilomètres.
Ismid	5	..
Lefké	80	105
Esk-Chéir	700	110
Sivri-Hissar	900	105
Plain of Abukar	1,000	220
Cesarea	1,100	190
Tunuz	1,200	155
Djarbékir	626	405
Mardin	592	110
Mossoul	250	294
Bagdad	50	483
Total		2,177 from Ismid.

The length of the several sections of proposed railway are as follows:—

	English miles.
Main line, Haidar-Pacha to Bagdad	1417
Branch from Eski-Chéir to Smyrna	331½
" " Mardin to Suedia	370
" " Scharkisla to Samsoom	237½
" " Aladjacham to Erzeroom	250
Total	2,606

of which 176 miles are already constructed and in traffic.

It is difficult to calculate the amount of traffic on these railways, but the Author estimates that it will be equal throughout to that which he has calculated will result on the Ismid and Eski-Chéir portion, and that it will be doubled in the course of ten years. The productions of the country are numerous, and will be exported in large quantities, but fuel for the locomotives will be very expensive. He estimates that if the railways are constructed with proper economy, admitting maximum gradients of 1 in 50, and curves of 330 yards radius, the average cost throughout being at the rate of £12,670 per mile for the 4 feet 8½ inches gauge, or £6,530 per mile for a narrow gauge railway, the total cost of all the above railways would be about £32,000,000 sterling for the broader gauge, or £16,400,000 for the narrow gauge. The lines from Ismid to Lefké, and the branches from Ada-Bazar and Sirvas to Samsoom, he considers would be paying lines from the first, but not so the remainder, which would require a subvention, or guarantee of interest, in order to make the construction possible.

O. C. D. R.

On the Cause of Failure of the Brakes on the Wädensweil-Einsiedeln Railway. By Hr. Pippart.

(Organ für die Fortschritte des Eisenbahnwesens, vol. xv., p. 95.)

In an official report prepared by Professor Sternberg on an accident which occurred on the Wädensweil-Einsiedeln railway, by a train breaking away on an incline, the cause is attributed to the fact that the engine driver, when he found the ordinary brakes ineffective, reversed the engine, but omitted to slacken the brakes, which thus were made to oppose the free backward motion of the driving wheels.

It appears from the evidence that the engine brake did not act sufficiently at the time of the accident on account of the brake blocks firing, whereby the coal dust or graphite on the burnt surface acted as a lubricant and disabled the brake. According to another account, some oil must have been splashed over the brake blocks which had the same effect in disabling them. This last account seems to be the more likely of the two, as coal dust has never before been known to act as a lubricant, nor is it clear how graphite could have been produced by the firing of the brake blocks; and from calculation the brake was powerful enough to skid the wheels, so that even a continued lubrication with coal dust or graphite could not have rendered it entirely inefficient. The real cause of the failure of the brakes was therefore some hidden defect in them, aggravated by accidental lubrication.

The failure of a brake in arresting the motion of a train may be due to two causes; first, the brake may not produce friction enough on the rim of the wheel to prevent the latter revolving, and secondly, even if the brake be powerful enough to skid the wheel, yet there may not be friction enough between the wheel and the rail to arrest the motion of the train. On this occasion both these causes occurred together; in the first instance, the ordinary brakes were found to fail in stopping the revolutions of the wheels, and subsequently, when the engine was reversed, the wheels were found to slip the rails, failing entirely to arrest the motion of the train.

With regard to Professor Sternberg's opinion, supported as it is by a series of mathematical formulæ, the writer only refers to the final conclusions arrived at. One of these is that, during the combined action of the brakes and the reversed motion of the driving wheels, there was a moment when the engine power overcame the friction both between brake and wheel and between rail and wheel; in consequence a quick and powerful rotation of the driving wheels took place, the masses to be put in motion being small, and the friction between wheel and rail becoming considerably less with the increased velocity. The writer admits that the action of the engine on the driving wheels after reversing may cause them to

slip, but he fails to see why the friction between wheel and rail should then become so much smaller; since it has always been accepted as an axiom that the co-efficient of friction does not alter with the velocity of the rubbing surfaces; and until Professor Sternberg supplies the proof of his assertion, the braking effect of revolving wheels will have to be taken as nearly the same as that of skidded wheels.

Professor Sternberg furthermore says that the driver, on reversing the engine, should not have put on full steam, as the engine was a very powerful one, intended to draw heavy loads up the incline, by means of the Wetli rack system; and that by so doing he caused the wheels to slip and revolve in the reverse direction, which would not have taken place had he only admitted sufficient steam to stop the forward rotation of the wheels, in which case the train would soon have been under control again.

The writer admits that the engine may have been powerful enough to make the wheels slip in spite of the ordinary brake being on, but he maintains that the friction between rail and wheel remained the same as before and did not become reduced to zero. The exact amount of this friction cannot now be ascertained, but since it was not able materially to reduce the speed of the train, the conclusion is that some lubricating material must have existed between the wheels and the rails; and it is evident that this lubrication would have rendered both the ordinary brake and that applied by steam power in reversing the engine ineffective, even if they were employed independently, which, according to Professor Sternberg, would have prevented the accident.

The writer holds, therefore, that the driver could not have acted better under the circumstances than by putting both descriptions of brake power into operation when one had failed, and therefore cannot be made answerable for the catastrophe.

W. R. B.

The Belpaire Steam Carriage for Railways.

By O. Dutillœul.

(Annales de l'Association des Ingénieurs sortis des Écoles Spéciales de Gand, 1878, p. 137.)

The steam-carriage designed by M. Belpaire for railway traffic, consists of an engine, boiler, and carriage-body on one frame. The body is separated by a passage from the engine, and is divided into two compartments, first and second class, seating each twenty-two passengers. Entrance to the first-class compartment is gained by a platform behind. The body is 9 feet 9 inches wide, and the seats run longitudinally, two at the sides, and two in the middle. The sides and ends of the frame are of iron, of U section, 10 inches

deep, and there are intermediate cross-bars of angle-iron. The total length of the frame is 35 feet 6 inches.

The boiler is of steel, of the usual locomotive type, having an additional cylinder or barrel added to it above, and connected to it. The grate is 24 inches wide and 28 inches long. The body of the boiler is 32 inches in diameter, and contains one hundred and forty-seven tubes, about ¾ inch in diameter inside, and 4 feet 1 inch long. The heating surface amounts to 237 square feet, for 4⅔ square feet of grate. The products of combustion are conducted along the upper surface of the boiler, and so dry the steam to a greater or less extent. The boiler is laid across the frame. The cylinders are horizontal; they are 7·20 inches in diameter, with a stroke of 11·20 inches. The vehicle runs on six wheels 3 feet 3 inches in diameter, on a base of 16 feet 7 inches. The leading wheels are the drivers, and as they are placed directly under the boiler, they carry half the total weight of the vehicle when loaded, which weighs nearly 21 tons, distributed as follows:—

	Without Passengers.	With Passengers and Attendants.
	Tons.	Tons.
Leading axle (driving) . . .	10·0	10·45
Middle axle	3·6	5·20
Trailing axle	3·7	5·30
	17·3	20·95

It is taken that, besides forty-four inside passengers, there are six more on the platforms, with a driver and a guard; in all, fifty-two persons. The coal-box holds about 7 cwt. of coal, which lasts for 124 miles. The water-tank, holding 250 gallons, is placed below the body. The brake is of the tramway-car type, worked from either end; it is applied to the hind wheels.

The consumption of coal averages 5·88 lbs. per mile. A speed of 45 miles per hour is attainable on the level; and 25 miles per hour on an incline of 1 in 62. Assuming a daily service of 124 miles, the working expenses would not exceed 10*s.* per day, or 1*d.* per mile run.

	s.	d.
Wages of engineman and guard	7	1½
Fuel, 7 cwt.	1	11¼
Oil, grease, water, &c.	0	10
Total for 124 miles . . .	9	10¾

The Belpaire Railway Steam-Carriage is likely to be much employed in suburban traffic.

D. K. C.

Iron Permanent Way for Street Tramways. By Hr. Böttcher.

(Zeitschrift des Vereines Deutscher Ingenieure, 1878, p. 270.)

In the system of permanent way introduced by the Author, the rails are supported by cast-iron chairs having a base of 18 inches by 10 inches, placed 5 feet apart, excepting at the rail joints, where they are 4 feet centre to centre, fish-plates being used 1 foot 7½ inches long. The chairs and rails are connected by screw-bolts, and kept in gauge by transverse tie-bars below the rails, and further stiffened on curves and at crossings, by a second set of tie-rods connected with the chairs near their base. In cases where the general street traffic is considerable, the rail between the chairs is in addition supported by a longitudinal row of paving stones, a channelled section of rail being used, the edges of which are rounded off so as to diminish obstruction to the transit of general traffic, and any accumulation of dirt or snow being displaced by the wheels of the tram-car. Where this channelled section of rail is adopted, the distance between the chairs may be increased to 6 feet 6 inches.

Should it be found necessary to change a rail, it may be effected in a very short time, as the chairs usually need not be interfered with further than by unscrewing the bolts by which the rails are attached, and it is next to impossible for these to get out of gauge. The mode of laying in the first instance is as follows. A length of rails, chairs, tie-rods, &c., is put together on the spot, and being adjusted to gauge is bodily lowered into position, holes to the requisite depth having been previously excavated for the chairs. The direction and level being ensured, gravel is punned into the space around the chairs, the rails are temporarily removed, a paving of stone cubes of about one-fifth smaller area of base than head is laid in concrete bedding, longitudinally between the chairs, the rails replaced, screwed up, and, lastly, the paving of the horse-track completed in a similar manner. A trial length, laid at the charge of the Bremen Tramway Company, is found to be successful in point of economy and easy travelling. The cost of rails, chairs, tie-rods, &c. delivered in railway trucks at Oberhausen is 15*s*. to 16*s*. per mètre run. The sections of rail described are already in use in Antwerp, Cologne, Düsseldorf, and Metz.

D. G.

Compressed Air as a Motive Power.

By P. Banneux.

(Annales des Travaux Publics de Belgique, vol. xxxiv., pp. 187 and 391.)

Mariotte's law is frequently employed in calculations where it is not admissible, a practice which leads to error, arising from the

fact that a gas cannot be compressed, under ordinary circumstances, without an augmentation of its sensible heat. The law of Mariotte only applies to cases in which time is afforded for the return of the gas to its initial temperature. Now, a volume of atmospheric air, at 68° Fahr., suddenly compressed to a pressure of 4 atmospheres, is raised in temperature to 329° Fahr., and, instead of assuming a volume of one-fourth the initial volume, according to the assumed law, the final volume by compression is really three-eighths, or 37·4 per cent. of the initial volume, according to the adiabatic law of Poisson. This law is indicated by the expression $n^{\frac{1}{k}}$, in which n is the number of times that the initial pressure is increased, and $k = 1 \cdot 41$ for air. The calculated fraction of volume, three-eighths, was approximately verified by experiment at Seraing, where the final volume amounted to 40 per cent. of the initial volume, instead of 25 per cent., as deducible by Mariotte's law.

The law of Poisson applies equally well to the measurement of the work expended in compressing and forcing air into a reservoir. For instance, M. Daxhelet, at Seraing, measured by indicator-diagram the work expended for one stroke of the piston in compressing air to five atmospheres, amounting to 28,273 foot-pounds. By Poisson's law, the amount is calculated to be 29,200 foot-pounds, which is remarkably near that of the indicated work. By Mariotte's law, it would have been only 22,892 foot-pounds.

The economical advantage of maintaining the temperature of the air constant during the operation of compression would apparently, in the example just given, be measured by a saving of about 25 per cent. of the work required for compressing and storing the air. M. Cornet proposed the injection of pulverised water into the midst of the heated mass, for the purpose of cooling the air, and approaching the more economical conditions of working under a constant temperature. About the same time, Dr. Colladon's pulverised water compressors were working at the St. Gothard tunnel, with so much success that in compressing air to 8 atmospheres, the rise of temperature did not exceed 27° Fahr., and the law of Mariotte applied approximately to the work expended. So long as thirteen years ago—say, in 1865—M. Gallez, engineer to the "Charbonnages belges," employed water by injection to keep cool and preserve the valves of the air-compressors used in one of the mines. A compressed-air locomotive was employed to draw the loaded coal-wagons up inclined planes about 1,200 feet in length. Compressed air, at 3½ atmospheres, was admitted into the cylinder, and cut off at two-thirds, expanding in the remaining third of the stroke. To prevent injurious refrigeration, the cylinder was surrounded by quick-lime which was watered. The quick-lime was renewed three times per day. The slaked lime was brought to the surface, and used for making mortar.

D. K. C.

Cost of Working Tramways by Compressed-Air Cars.

By M. Mékarski.

(Résumé de la Société des Ingénieurs civils, 1878, p. 210.)

Recalling the results of his previous estimates,[1] M. Mékarski deduces that, by a cubic mètre of compressed air, at 25 atmospheres and at the average temperature 15° C., or 59° F., weighing 30 kilogrammes, the work that may be done per kilogramme of air amounts to 13,300 kilogrammètres: that is to say, 1 cubic foot of air at 25 atmospheres weighs 1·872 lb., and the work that may be done per lb. of air, amounts to 42,624 foot-pounds. And, as the frictional resistance on tramways is often above 10 kilogrammes per tonne, or 22½ lbs. per ton, the expenditure of compressed air is at the rate of 1 kilogramme per tonne per kilomètre, or 3·6 lbs. per ton per mile-run.

It has been proved that compressed air at 30 atmospheres can be produced by suitable apparatus, with a rise of temperature not exceeding 45° F., at the rate of 6 kilogrammes or 13¼ lbs. of air per HP. per hour. Allowing a consumption of 1½ kilogrammes, or 3·31 lbs. of coal per HP. per hour, by steam-power, the consumption for 1 kilogramme of compressed air is ¼ kilogramme of coal; or, for 1 lb. of air, ¼ lb. of coal per HP. per hour. It follows that, on tramways, it may be considered that compressed-air engines consume $(3 \cdot 6 \times \frac{1}{4} =)$ 0·9 lb. of coal per ton per mile on a level.

The cost of plant, buildings, and rolling stock is estimated on the assumption that each compressed-air engine travels 100 kilomètres, or 62 miles, per day on duty, resting two days a week, and making 25,000 kilomètres, or 15,500 miles per year—equivalent to an average of 70 kilomètres, or 43½ miles per day. According to the contract prices for the material, it appears that, for compressed-air cars, with stationary engine-power, and plant, the cost per car amounts to £1,360; and that, for a system of independent compressed-air engines, the total cost per engine is £1,800.

In comparison with these estimates, the total cost for omnibuses, horses, harness, and stabling, per omnibus of thirty-five seats, amounts to £1,180; and to £1,480 per omnibus of from forty-five to fifty seats.

The total cost for omnibus service, by the accounts of the Omnibus Company of Paris, amounts to 10·115*d.* per mile-run; and that for tramway service by horses, amounts to 7·356*d.* per mile-run. For the service by compressed-air propulsion, the annual working cost, including a charge for interest on the excess capital cost, is estimated at 4·60*d.* per mile-run, if compressed-air cars be employed, or 6*d.* per mile-run for separate compressed-air engines. Thus, an economy of 38 per cent., and of 40 per cent.,

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[1] *Vide* Minutes of Proceedings Inst. C.E., vols. xliii., p. 382, and xliv., p. 254.

respectively, can be effected, comparing the compressed-air cars with the present tramway service, and the separate air-engines with the present omnibus system.

M. Mékarski's system is shortly to be put in operation in Nantes and in Paris.

D. K. C.

Paris Omnibus and Tramway Company.

(Annales Industrielles, July 28, 1878, col. 98.)

This company works, in Paris, fifteen lines of tramway and thirty-three lines of omnibuses:

Total length of tramways	63·03	miles.
„ „ omnibus lines	126·72	„
Together	189·75	

The longest and shortest tramways routes are—

Longest	The Louvre to Saint-Cloud	6·28	miles.
	The Louvre to Charenton	5·31	„
Shortest	La Villette to the Place du Trône	2·85	„
	The Louvre to the Pont d'Jena	3·03	„

The average speed on the tramways is		5·32	miles per hour.
Highest speed	Saint-Cloud	6·28	„ „
	Jena	6·05	„ „
Lowest speed	Trône and Montrouge	4·94	„ „
	To the Eastern railway	4·44	„ „

The longest and shortest omnibus lines are:—

Longest	Panthéon to the Place Wagram	5·25	miles.
	La Villette to St. Germain-des-Près	4·65	„
Shortest	Louvre to Belleville	2·41	„
	Palais-Royal to Port Rapp	2·31	„

The average speed of the omnibuses is		4·84	miles per hour.
Highest speed	Port-Maillot to Hôtel de Ville	5·25	„ „
	Madeleine to the Bastille	5·36	„ „
Lowest speed	Montmartre to Port-Royal	4·31	„ „
	Clichy to the Odéon	4·39	„ „

For the service of all the lines, the company has fifty-two offices for the tramways, and one hundred offices for the omnibuses.

D. K. C.

Feed-Water and Incrustation of Steam Boilers.

(Annales Industrielles, July 1878, col. 31.)

At the Congress of the Chief Engineers of Steam-Users' Associations, held at Brussels in 1877, M. Roland referred to instances of boilers, the plates of which had been greased, and which had been free from incrustation, as well as from corrosion, whilst, on the contrary, the slide-valves and cylinders of the engines became corroded. M. Vinçotte preferred the application of a thin coat of coal-tar, instead of grease, for the prevention of incrustation; although, when the tar is thickly applied, there is a liability to the formation of agglomerations of loose crusts, and to consequent overheating. M. Bour stated that the plates could be very thinly coated by a mixture of light tar-oil and tar; but that the mixture might form a detonating compound within the boiler. M. Walther-Meunier described an instance of corrosion of the plates by grease; and M. Vinçotte ascribed bulgings and ruptures of angle-irons, as well as leakages, to the use of greasy water, which were entirely obviated by the employment of carbonate of soda in sufficient quantity. In another boiler, fed with non-calcareous water, a considerable quantity of grease had been collected, which gave rise to corrosion of the plates. But the employment of greasy water did not in all instances give rise to injurious effects. M. Cornut suggested that the difference of the effect might be due to difference in the composition of the greases used.

The process of purification of feed-water, introduced by MM. Béranger and Stingl, consists of three operations: 1, The chemical treatment of the water, for the precipitation of impurities; 2, The decantation of the water, in order to separate at least the greater proportion of the deposit; 3, The filtration of the water after having been decanted.

D. K. C.

Compound Steam Engines compared with the Corliss Engine.

By M. Fréminville.

(Annales Industrielles, July 1878, col. 86.)

At a conference held by M. Fréminville, in which he noticed the steam-engines at the Exhibition of 1878, he traced the history of the successive improvements in the steam engine; and, remarking that the greater the degree of expansion, the greater is the degree of refrigeration in the cylinder, he referred to the expedient of closing the exhaust before the end of the return-stroke, and the compression of the exhaust steam, as a means of partially counteracting that kind of loss, and of obviating the loss by clearance-space in the cylinder. He maintains that the efficiency of the engine may

often be considerably augmented by a compression of the exhaust steam to a suitable extent; for "the more the compression, the more the regimen of the engine approaches to that corresponding to Carnot's cycle."[1]

Engineers, unable completely to annul the loss by clearance-space, adopted the system of four valves employed by Corliss and others to such effect that, whilst M. Farcot, with a clearance amounting to 5 per cent. of the volume of the cylinder, has not reduced the consumption of coal below from $1\frac{3}{4}$ lb. to 2 lbs. per indicator HP. per hour; with the four valves, the clearance-space has been reduced to 2 per cent., and sometimes to one per cent., and the consumption of fuel has been brought as low as $1\frac{1}{2}$ lb. (1·54 lb.)[2]

By the employment of compound cylinders, the loss of efficiency by clearance-space may, in the opinion of M. Fréminville, be to a great extent removed by a suitable adjustment of the slides, such that the intermediate pressure in the interspace between the cylinders may be equal to the pressure of the steam as exhausted from the first cylinder. In some instances, compound cylinders have been fitted with four valves on the first cylinder; but the economy resulting from this application has not amounted to more than 2, 3, 4, or 5 per cent., at the most.

D. K. C.

Efficiency of Steam Worked Expansively, in the Compound Engines of the "Cher."

By B. F. ISHERWOOD, Chief Engineer U.S.A.

(Journal of the Franklin Institute, July, August, 1878, pp. 1, 73.)

The experiments, the results of which are investigated by Mr. Isherwood, were made in 1866, with the engines of the French naval steamer "Cher," at Cherbourg. The vessel was held stationary, and the power applied to the displacement of water by the screw. The engines were of the Dupuy-de-Lôme type, having three cylinders of equal diameter, 43·30 inches, and stroke, 1·64 feet, connected to one crankshaft:—the two outer cylinders by cranks at 90°, and the middle cylinder by a crank at 135°, with each of the outer cranks. The steam from the boiler was cut off at 88 per cent. of the stroke of the middle cylinder; whence it expanded simultaneously into the two outer cylinders, in which it was cut off at 78 per cent. of the strokes. From these the steam was exhausted into surface-condensers. The admission to the middle cylinder could be varied by an expansion-valve. The

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xlii., p. 336.
[2] *Ibid.*, vol. liii., p. 365.

cylinders were not jacketed. There was one Mangin screw, having two pairs of blades, 11·48 feet in diameter.

The experiments were made to test the efficiency of superheated steam, and the influence of the boiler-pressure, without wiredrawing, on the efficiency; also the influence of wiredrawing; and, finally, the efficiency of saturated steam admitted to the outer cylinders only. The experiments were arranged by the Author in five series, the results of which have been very fully stated and tabulated. These are given in abstract in the following table:—

—	First Series.	Second Series.	Third Series.	Fourth Series.	Fifth Series.
Number of trials	4	4	4	2	1
Average boiler pressure . lbs.	27·5	19·5	27·3 to 12·2	27·9	18·5
Cut-off at . per cent.	88 to 40	88 to 40	88	88	78
Actual ratio of expansion . .	2·3 to 4·6	2·3 to 4·6	2·26	2·26	1·26
Turns per minute	66·3 to 54·6	58·8 to 48·1	66·3 to 51·2	59·7 and 53·7	66·3
Degrees of superheat. Fahr.	(av.) 14·1	(av.) 21·8	11·5 to 30·3	18·1	—
Indicator HP. .	492·1 to 269·9	337·5 to 189·1	492·1 to 224·5	340·1 & 267·2	491·6
Cold water consumed per Indicator HP. per hour . . lbs.	23·52 to 22·42	25·12 to 23·74	23·52 to 26·40	24·11 & 26·16	33·66

From the results of the first and second series, taken together, as detailed by the Author, it appears that, for an expansion of from three times to five times, in a compound engine, without a steam-jacket, with superheated steam, the measure of expansion does not affect the economy of the steam in the production of horse power. Comparing together the first and the second series, it appears that, though, for the total horse powers equal quantities of water were consumed per horse power, for the indicator power, the consumption per horse power was greater for the lower pressures in the second series, even though the steam was superheated by a greater number of degrees. In the third series, the consumptions increased as the boiler-pressure was lowered and the speeds reduced, but the speed in itself did not necessarily affect the consumption. For the total horse power, the consumption per horse power was nearly constant. In the fourth series, the steam was wiredrawn, and, under the circumstances, the consumptions were about the same as when steam of full pressure was admitted. In the last experiment, saturated steam was used at a low pressure, and expanded only 1·26 times. The result was a great increase of the consumption.

A few particulars of fuel consumed are given in the Paper.

D. K. C.

On Steam Engines with variable Expansion Gear.

By H. De Wilde.

(Annales de l'Association des Ingénieurs sortis des Écoles spéciales de Gand, 1877–78, pp. 57, 69.)

These are the first two of a series of Papers on the expansive working steam engines of the Corliss class, the construction of which is a speciality of engine manufacture in Ghent. The horizontal engines constructed by M. Charles Nolet are the subject of this Paper. Briefly sketching the differences of the old and the new systems, the Author notices that, whilst, in the old engines, steam was admitted for a constant fraction of the stroke, and the pressure was varied by means of the governor and the throttle-valve, in the new engines the pressure is constant, and the period of admission is varied, for the power, by the governor. Wire-drawing of steam during its admission into the cylinder is obviated by the prompt and almost instantaneous opening and closing of the valves, for which purpose springs are employed in nearly all the new systems of valve-gear. The prompt action of the valves became the more necessary since the speed of the piston had been increased from 200 feet per minute in the old engines to 400, 600, or even 800 feet per minute, which was done with a view to reducing the dimensions and loss of heat, and was effected by augmenting the length of the stroke without excessively increasing the number of revolutions in a minute. Since the introduction of engines of the new system, the power required for large factories has, in many instances, been derived from a single steam engine, where formerly several engines may have been employed.

Another difference consists in the great reduction of the clearance-spaces in the cylinders, and in the employment of a degree of compression sufficient to raise the pressure of the exhaust steam to the same pressure as that in the boiler. The steam passages are distinct from the exhaust passages; thus the condensation of the steam from the boiler is diminished. The exhaust passages are large, and freely carry off condensation water from the cylinder. The feed-water is heated in a coiled tube, on the outside of which the exhaust-steam is delivered. Finally, the steam-jacket is employed, and superheated steam is used.

In the construction of engines, the simple direct frame has been introduced, uniting directly the cylinder and the crank-shaft bearing. In horizontal engines of great power the piston-rod is prolonged, and takes a bearing in the other cylinder-cover, to relieve the cylinder of the weight of the piston, and to work the air-pump. The brasses of the crank-shaft are made in four segments, adjustable for wear, and renewable without lifting the shaft. The connecting-rods are made solid at the ends; the pistons are very simple, without either bolts or screws. The piston-rods, the cross-head, and the crank-pin are of steel; also in some cases

the crank-shaft. Toothed fly-wheels are replaced, in many cases, by belt fly-wheels, which work more smoothly. For the admission of steam to the cylinder, the ordinary flat valve, the oscillating valve, or the equilibrium valve, is employed; for the exhaust, one of these, or a gridiron valve, is employed. The prime condition of a valve is tightness.

In the horizontal engine by M. Nolet, the cylinder is solidly and directly connected to the crank-shaft bearing by a strong frame of cast-iron, which serves also as a guide to the cross-head. The cylinder and the axle-bearing are each placed on a foundation of mason-work; between them are placed the feed-water heater, the condenser, the air-pump, and the feed-pumps. The pumps are worked by a special connecting-rod and levers. At each end of the cylinder, there is a balanced valve for the admission, and a grid for the exhaust, worked by cams. Balanced valves for admission of steam are very efficient. They admit the steam, and they cut it off, without wiredrawing; well made, they are perfectly tight when closed, and they last long in good order.

The lift of the valve is not more than from $\frac{3}{4}$ inch to $1\frac{1}{8}$ inch; and, with the aid of the dashpot, the valve may return to its seat without a blow. The tightness of such valves, even after several years of work, has been tested and proved by direct trial. The slide-valve has, for several years, been generally abandoned: it has been found wanting in steam-tightness.

The piston is a simple disc, having three, four, or five circular grooves in the circumference, in each of which is a cast-iron packing-ring, which acts as a spring and makes the piston tight. The steam is cut off at from $\frac{1}{8}$ to $\frac{1}{6}$, with steam of five or six atmospheres in the boiler. The cylinder is not steam-jacketed.

D. K. C.

Steam-dredging Machine at Toulon. By M. Hersent.

(Annales Industrielles, July 14, 1878, col. 42.)

For the works of the Missiessy wet dock at the Port of Toulon, it was necessary to increase the depth under water from 33 feet to 59 feet, by excavating in a very tough soil, known at Toulon as *safre*, consisting of an agglomeration of flint-stones, clay, and sand, with beds of calcareous conglomerates.

The hull of the dredger is 108 feet long, 21 feet 4 inches wide, $8\frac{1}{2}$ feet deep, having a draught of 3 feet 7 inches. The driving power is supplied by a high-pressure expansive surface-condensing steam engine, developing 50 HP. The winches are worked by an engine of 6 HP. These winches are on the double-drum system, working the chains by friction. The bucket-frame consists of a pair of booms or jib-poles (*élindes*), on which the buckets are extended. It is 82 feet in length, and is available for dredging to a

depth of 59 feet. It takes a free bearing on a transverse beam in the well of the hull, and may be placed in a vertical position without shifting the position of the buckets with respect to the discharging troughs. A guide-roller is placed in the well, by which the chain of the bucket is maintained at a constant inclination, whatever the angle of the frame. This is an important feature, and by its introduction the management of the machine is greatly ameliorated. It relieves the tension on the bucket-chains when much inclined. The chain is driven by a four-sided tumbler at the upper end, and it passes under a six-sided tumbler at the lower end of the boom.[1] The buckets are of steel, and have a capacity of 9 cubic feet. Pickers are fixed to the chain, alternately with the buckets, in order to loosen the ground preparatory to the action of the buckets. By these pickers, stress on the chain is made more nearly constant. The chain is guided during the ascent, on rollers carried by the frame.

In practice, the dredger removes about 650 cubic yards per day of ten hours, when the upper tumbler makes six revolutions per minute. At this rate, it appears that the buckets are filled to one-third of their capacity for the time they are in action, or to one-fourth for the total time.

The barges for the reception of the material have a capacity of about 65 cubic yards.

D. K. C.

Wonlarlarsky's Transhipping Crane.

(Revue industrielle, 1878, p. 355.)

This apparatus is employed for the unloading and removal of merchandise and other material when the distance is too great for the employment of a fixed crane. It is available for removing weights not exceeding 8 cwt. for distances extending to 100 or 120 feet. The apparatus is composed of iron joists, rolled or riveted, or of timber joists, 20 feet or 25 feet long, joined end to end to form a gangway on which a pair of rollers may run, weighted with the load to be carried. Each joist is suspended at a suitable height above the ground—10 or 12 feet—by a triangle about 21 feet high, by means of differential pulleys. The joists, joined end to end, form a continuous gangway, laid at such an inclination that the rollers with their load may run down by gravitation to their destination. The goods are lifted by means of winches to the upper end of the incline, where they are attached to the rollers. The apparatus can be worked by three or four men. Since 1872, this apparatus—the *grousokate*, as it is called—has been much employed for the movement of Russian artillery in maritime ports.

D. K. C.

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[1] In the text this tumbler is stated to have eight sides.

The New Man-engine at Přibram. By V. Mayer.

(Oesterreichische Zeitschrift für Berg- und Hüttenwesen, vol. xxvi., pp. 203–215.)

The man-engine in use in the Maria shaft at Přibram at present extends to the twenty-fourth level, or 708 mètres (2,322 feet) below the surface; but the mine, opened out, and ore workings have been commenced from four lower levels, the lowest, or twenty-eighth, being 900 mètres deep, and farther sinking down to 1,000 mètres is in progress. It has therefore become necessary to make provision for an extension of the engine to a depth of 1,000 mètres at least, within a short time, or an addition of 292 mètres (957 feet) to the present length of rods.

As it is desirable upon the score of economy to keep the moving weight down to the lowest possible point, in order to preserve the coupling-rods, bell-cranks, and other moving parts at present in use, it has been decided to substitute an entirely new rod, which, for the whole length of 1,000 mètres, shall be no heavier than the present one of 702 mètres. This can be done by using Bessemer steel of a tensile strength of 65 kilogrammes per square millimètre (41 tons per square inch) instead of iron; the former metal has therefore been selected for the new engine. This is to be made in lengths and solid throughout; the method formerly adopted, of building up the rod from a series of flat bars (three to six according to the section required) laid side by side and united by fish-plates and through-bolts, being considered to be not only expensive but objectionable in many ways. More particularly it is found that from irregular motion in the bolt holes, the strains on the constituent bars are not uniformly distributed, and consequently fractures from overloading are not uncommon. The replacing of a fractured plate is, however, a very difficult matter, and in some cases is practically impossible.

In order to obtain uniformity in the quality of the metal, the rods are to be forged and not welded. The section, a rectangle whose sides are in the fixed proportion of 1 to 2 throughout, is reduced in dimensions at every division of eight rods. In the lowest, or thirty-third division, where the rods are smallest, the sectional area is 560 square millimètres (·868 square inch), the breadth being 33·4 millimètres, and the thickness 16·7 millimètres; while in the first, or uppermost, the dimensions are—breadth, 95·4 millimètres; thickness, 47·7 millimètres; area, 4,541 square millimètres (7 square inches). The minimum section of the bottom series is somewhat larger than that theoretically required, partly in view of a possible extension in depth, and partly to provide for a certain reduction in area by rusting. The rods of the present engine, which have been in use since 1867, are much rusted throughout. At the end of each rod a projecting square head is forged, over which are placed a pair of claw-ended coupling-plates secured by four through-bolts. The butts of the

rods are not in contact, but are separated by a hollow space of 10 millimètres, into which steel wedges are driven. The platforms are trapeziform, of sheet-iron 4 millimètres (0·157 inch) thick, and supported directly upon the head of the couplings; they are of sufficient size to carry two men at a time, as the experience obtained from the engine at present at work shows that men will travel in pairs although forbidden by regulation to do so. The rods are connected by counterpoise chains passing over rollers placed at intervals of eight platforms. The connections for these as well as those of the hand staples and other fittings of all kinds are made by overlapping pieces, so as not to weaken the rods by screw holes or other perforations.

The following are the leading dimensions adopted in designing the work:—

Length of rod between platform . . .	7·586 mètres.
„ stroke	3·793 „
Weight of a platform	40 kilogrammes.
„ pair of counterbalance chains .	130 „
„ a catch-piece	56 „
„ a guide	25 „
„ a man	70 „

Maximum load 8 kilogrammes per millimètre, or about ⅙ of the breaking strain.

Every section of four rods is provided either with a catch-piece or a pair of balance chains, and below 500 mètres with a guide to prevent lateral oscillation. The additional weight due to the couplings is about 10 per cent. of that of the rods.

The total weight upon the pins by which the rod is attached to the bell crank or quadrant is, when the former is fully loaded with men, rather more than 56½ tons, made up of the following items:—

	Tons.	Tons.
One line of 1,000 mètres of rods . . .	29·711	
Sixteen connecting chains	2·080	
„ catch-pieces	0·996	
Seventeen guides	0·425	
One hundred and thirty platforms . . .	5·200	
		38·412
Two hundred and sixty men		18·200
Total	Tons	56·612

The weight of the entire engine (both rods), when not loaded with men, is 76·824 tons.

The bell cranks are made with arms of unequal length, 4·740 mètres and 3·950 mètres, to avoid the use of an excessively long crank; they are of a box construction in sheet-iron, with cast-iron bosses for the shafts and connecting pins, and weigh 6·4 tons each; the connecting rods are of steel. The motive power is furnished by a horizontal steam engine of 560 millimètres (22 inches) diameter of cylinder, and 1·110-mètre (43·7 inch) stroke, working with variable expansion.

H. B.

The Water-pressure Engines at Clausthal. By Hr. Fickler.

(Zeitschrift für das Berg-, Hütten- und Salinen-Wesen, vol. xxvi., p. 233.)

The completion of the new deep adit from Gittelde at the base of the Harz Mountains at Clausthal, a distance of 14¼ miles, which receives the drainage water of the mines at depths varying from 334 to 370 mètres below the surface, has rendered a reform of the pumping arrangements necessary, in order to obtain the full effect of the released lift, in pushing the working to greater depths. The bottom levels at present are 710 mètres (2,329 feet) deep. Hitherto the pumping has been mainly done by water-wheels, working 5 to 9-inch bucket pumps in lifts of 26 feet, by means of sweep rods, quadrants, and wooden rods in the shafts.

Fifteen wheels were in use in different mines, partly above and partly below ground. The natural falls of water, after doing duty at the surface, were taken down to the shafts to some of the numerous adits, producing fresh falls, which were, however, very imperfectly utilized. Thus in the St. Elizabeth shaft, with a disposable volume of rather more than 1,000 gallons per minute, with a total fall of 291 mètres (954 feet), only 50 mètres were actually used, and in another case, only 30 mètres out of 161 mètres.

The total length of pump rods in use was about 10,000 mètres (6¼ miles), and that of the pump lifts, 2,680 mètres. The maintenance and supervision of such a quantity of rods was exceedingly costly, as, in consequence of the great and increasing depth of the mines, the pumps were constantly overloaded, with the natural result of numerous breakages. For in 1876, 575 ten-mètre lengths (more than 3½ miles) of rods required renewal; and in dry seasons, from the supply of water failing, the mines were in some instances drowned to a height of 130 mètres above the lower workings, and required a considerable expenditure of time and cost, on the return of more favourable conditions, to pump them out again.

The first steps towards an alteration in the system of the pumping were taken in the year 1856, when the Queen Mary shaft was commenced, in anticipation of the completion of the new adit. It is so placed as to receive water from three reservoirs, at a point 20 mètres below the surface, with a useful fall of 368 mètres to the adit; the quantity available being 25 cubic mètres (882 cubic feet) at periods of maximum flow, and 2·5 cubic mètres (88 cubic feet) in the driest seasons. The bottom workings, 203 mètres below the deep adit, are to be connected, by a so-called "deepest water level," with all the mines of the district, a length of about 100 mètres having been purposely left uncompleted until the new engines were ready for work. The original plan provided for two direct-acting water-pressure engines with hydraulic counter-balances, placed 103 mètres below the adit, and making four strokes at most per minute. In 1866, on the completion of the shaft, however, other ideas prevailed, and a

new plan was elaborated for the use of rotatory engines placed at the bottom of the mine, and forcing the water at one lift to the adit, without the intervention of shaft rods, which has been carried out. Two pairs of coupled horizontal engines, with double-acting plunger pumps and fly wheels, are placed in chambers 20 mètres long and 10 mètres wide, right and left of the shaft, about 5 mètres above the bottom water-level. The two driving columns, one for each pair of engines, are carried in the main shaft down to the fifth level, 492 mètres from surface, where they pass by a short branch level to a smaller shaft, 2·88 mètres long by 2·16 mètres wide, down to the engine level. They are placed close to one of the shorter sides of the shaft, the two rising columns for water discharged occupying a similar position on the opposite side. The latter are connected by another short horizontal branch at the fifth level, with another shaft of smaller dimensions containing two similar columns, which extend up to the point of final discharge at the adit. Thus no portion of the pumping machinery is contained in the main travelling and winding shaft below the 492-mètre level.

The tubes forming both series of columns are put together in 3-mètre lengths, with flanged joints, packed with linen and india-rubber rings, sliding telescopic compensation joints, with leather packings, being introduced at several different levels, to allow for variations in the temperature of the driving water at different seasons of the year. The pressure pipes are 225 millimètres (8·8 inches) inside diameter down to the fifth level, and 185 millimètres (7·2 inches) below it, the larger dimensions being adopted above to allow of a portion of the supply being taken off for driving other machinery at a future time. The rising pipes, which take the discharge both of the engines and the pumps, are of 275 mètres (10·8 inches) bore. The thickness of metal in the pipes is 15 millimètres at the top of the pressure column, increasing by 5 millimètres at each compensation joint downwards, to 40 millimètres at the fifth joint; below this, with the diminished internal diameter the substance is reduced to 35 millimètres down to the last compensation joint, where it is again increased to 40 millimètres. The rising pipes are 30 millimètres thick at the bottom, and diminish similarly upwards by differences of 5 millimètres to 15 millimètres. The proof strain varies according to the thickness and diameter from 32 to 122 atmospheres, the amount of failures under test was very small, being less than 2 per cent.

The engines are of exceedingly simple construction; the pistons are cylindrical plugs, whose length is double their diameter, with six grooves, square edged round the circumference, cast solid with the rods, the same construction being used both for the engine and pump pistons. The piston rod passes through both ends of the cylinder, and is connected on one side with that of the pump, and on the other with the crank and fly-wheel. The valves, which are moved by eccentrics on the fly-wheel shaft, are a pair of solid cylinders on the same rod, moving in short lanterns placed in the openings of the cylinder ports. These lanterns are perforated

with six lenticular openings, which, being alternately open and shut by the passage of the valve, form the water-way from the pressure column to the cylinder, and thence to the discharge-pipe. To prevent injurious shocks at the change of stroke, these passages are made a little longer than the depth of the valve, so that when the latter is in the middle position, corresponding to the end of the stroke in the cylinder, the ports are not completely closed, and both sides of the piston are for the moment in communication with the pressure column, *i.e.*, they are in equilibrium. This contrivance is, of course, attended with a not inconsiderable waste of water—which, however, is of small consequence, owing to the supply being more than sufficient for the requirements of the engines—but it has the advantage of dispensing with the use of air-vessels or relief-valves.

The cylinders, pistons, piston-rods, valve-seats, and cocks are made in a bronze of the following percentage composition: Copper, 89; tin, 5½; zinc, 5½; the bearings of gun-metal, and the connecting rods, eccentrics, cranks, fly-wheel shaft and arms, of wrought iron. The pistons are not packed, and although very carefully fitted, are not improbably supposed to allow a considerable amount of water to pass through them, owing to the high working pressure (from 800 to 900 lbs. on the square inch). The stuffing-boxes are packed with alternating roughened brass and leather discs. The amount of water to be lifted is 2·5 cubic mètres (about 550 gallons) per minute, each pair of engines being equal to three-fourths of the work. The following are the principal dimensions:

Total hydraulic head	597·272	mètres.
Negative head (hydraulic balance)	228·872	„
Effective working head	368·40	„
Stroke in cylinder and pump barrel	0·625	mètre.
Mean speed of ditto	0·25	mètre per second.
Volume of power water	1·6	cubic mètre per minute.
„ of discharge	1·878	„ „
Diameter of cylinder	0·310	mètre.
„ of pump	0·328	„
„ of piston rod	0·160	„
Working surface of piston	0·0533	square mètre.
„ „ of pump	0·0623	„ „
Pressure on each piston	19,635	kilogrammes.
Length of connecting rods	3·060	„
Radius of crank	0·312	mètre.
Mean radius of fly-wheel	4·7	mètres.
Weight of rim of „	8,000	kilogrammes.
Diameter of valve cylinder	0·174	mètre.
Depth of valve	0·180	„
Depth of water passages in valve lantern	0·200	„
Length of stroke of valve	0·400	„
Diameter of pipe stop-valve on admission	0·150	„
„ „ „ on discharge	0·225	„
Maximum working speed	12	revolutions.
Gross power	130·5	HP.
Nett „ at 75 per cent. duty	97·87	HP.

The admission-valve, as originally constructed, soon became unworkable, owing to irregular strains, and has been replaced by

one of double beat form, of 145 millimètres diameter, with 12-millimètre bearing surfaces of seat.

The weight of metal in the different parts of the engines is: of cast iron, 118·44 tons; of wrought iron, 22·63 tons; and of bronze and gun-metal, 39·92 tons; and in the pressure and rising columns: cast iron, 358·5 tons; and wrought iron, 10·45 tons.

The work was let by contract, eighteen tenders having been submitted, at prices varying from £14,227 to £26,210; the designer's estimate being £19,270: the lowest tender—that of the Cologne Joint Stock Engine Company at Bayenthal—was accepted.

The erection of the pressure tubes was commenced in November, 1875; the southern pair of engines was started December 15, 1876, and the northern pair, on the 15th of May following.

The entire cost, exclusive of the mining works proper,[1] but including all foundations, has been £14,949; of which amount, £13,325 are chargeable to the construction and erection of the engines, and £1,624 to the foundations and arches, and bearing timber on the shafts.

The engines work without shock up to sixteen revolutions per minute. Until the communication with the neighbouring mines is completed, the water is kept under by sixteen hours' daily pumping, at seven strokes per minute. One man is in attendance in each engine-room, making a twelve-hour shift.

Theoretical Considerations on the Clausthal Water-Pressure Engines. By Hr. Hoppe.

(Zeitschrift für das Berg-, Hütten- und Salinen-Wesen, vol. xxvi., p. 240.)

The memoir contains a complete mechanical analysis of the construction of the various details of the Clausthal water engines; the theoretical duty at different strains; loss of head, &c., and a determination of the actual duty by gauging. From the latter it appears that the effective work is far behind the theoretical. The results obtained at different speeds are tabulated as follows:—

Number of Revolutions per Minute.	Measured Volume of Water Expended.	Volume described by Pistons in Cubic mètres. = 35·316 Cubic feet.		Measured Volume of Water Lifted.	Gross Power of Water.	Effect realised on Pumps.	Percentage duty.
		Engines.	Pumps.				
	Cubic mèt.			Cubic mèt.	HP.	HP.	
3	1·700	0·404	0·471	0·410	139·2	20·9	15
6	2·008	0·806	0·943	0·800	164·4	40·7	25
9	2·339	1·211	1·414	1·121	191·5	57·0	30
12	2·700	1·615	1·886	1·500	221·0	76·3	35

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[1] These appear from the plate to include two engine-rooms, each 2,000 cubic mètres volume, a shaft of 6·2 square mètres section, 128 mètres deep, another 4·15 square mètres section of 100 mètres deep, and about 72 mètres of levels.

The absolute loss of water in the engines is nearly constant—from $1 \cdot 1$ to $1 \cdot 3$ cubic mètres for all speeds; and varies proportionately, from $\frac{13}{17}$ at three revolutions, to $\frac{10}{27}$ at twelve revolutions, of the total quantity expended; about one-third of this is due to the non-closing of the equilibrium passage, and a great part of the remainder to the absence of piston packings. The friction brake trials, which it was proposed to make, have not been carried out, owing to the breaking of a crank in one of the engines. The discharge useful effect of the pumps is from 80 to 87 per cent. of the theoretical volume.

H. B.

Waterwheels. By A. Bonolis and F. Mazza.

(" Delle Ruote idraulice ad asse verticale e più specialmente delle ritrecini.") Tract. 8vo. Napoli, 1878.

The object of the pamphlet is to show that the class of water-wheels known in Italy as "ritrecini" are capable, when properly constructed, of an efficiency of fully 60 per cent. To this end the Authors give the results of official experiments, made, for fiscal purposes, on the wheels of thirteen flour mills in the commune of Gragnano, near Naples. These show a coefficient of efficiency varying from $0 \cdot 40$ to $0 \cdot 65$, according to the circumstances under which they were made. The maximum in each case lay between $0 \cdot 60$ and $0 \cdot 65$ (in one instance, indeed, it reached $0 \cdot 70$), and was always obtained when the velocity of the centre of percussion of the vanes of the wheel was about half that due to the head of water.

The wheels are set with their shafts vertical, and, when made in the most approved form, consist of two concentric cylinders about 4 inches deep, between which are fixed from forty to sixty vanes, each about 4 inches deep by 6 inches wide. These vanes may be inclined or vertical, and have generally projections on their upper and lower edges about 2 inches deep, so as to form boxes or buckets, it having been found that a better result is obtained when the water is discharged into these than when it simply dashes against a plain surface. In some cases only the bottom ledge is added, while in others the vanes are curved; their form seems to have no effect on the efficiency. In some localities the cylinders are dispensed with, and the vanes are made in the shape of spoons, and fixed to the ends of the arms of the wheel.

The diameter, measured from centre to centre of the vanes, is usually 5 feet, or a little more, and the weight of the wheel varies from 30 to 40 lbs. The number of revolutions varied in the experiments from 60 to 130 per minute. The water is delivered from the reservoir into the wheel by a pyramidal or conical mouthpiece, inclined at a small angle to the horizon, in a vertical plane tan-

gential to the circle described by the centre of percussion of the vanes.

In designing new wheels the Authors admit that calculations based on theoretical considerations are of little value, and recommend that the general form be copied from such wheels as are working with satisfactory results, the number of revolutions and the area of the vanes being determined by means of the ratios:

$$\frac{v}{V} = c \text{ and } \frac{Q}{s\,v} = k,$$

in which

v is the velocity of the centre of percussion of vane;
s its surface;
V the velocity due to the head on the orifice of the discharge pipe;
Q the actual discharge.

The most advantageous values of these ratios should be determined by experiment. The figures given in the paper prove that, for the class of wheels to which they refer, the most advantageous value of c lay between 0·48 and 0·55; the corresponding value of k varied from 0·18 to 0·40, but for most of the wheels it was about 0·20. The cost of a wheel is said to be about £2.

M. L.

On Force-Pumps with Tubular Rods. By Prof. H. Undeutsch.

(Civilingenieur, vol. xxix., p. 267.)

The Author describes various arrangements of pumps for mining purposes, in which the delivery-pipe, or a portion of it, is used as a pump-rod, and he discusses and compares their merits and demerits from a theoretical and practical point of view.

He states the formulæ for determining the principal dimensions, weight, &c., for various constructions. Preference is given to Rittinger's pump, as being the cheapest, most economical in working, and taking up the least room in the shaft. In Rittinger's pump the lower end of the delivery-pipe acts as the plunger, in the interior of which the delivery valve is placed; the uppermost portion, from which the overflow takes place, is rigid, and fits into a stuffing-box at the end of the movable part forming the pump-rod. Another modification is also described by the Author. The firm of Hoppe in Berlin has, for many years, made pumps of the above description, from 1,000 HP. downwards, for various mines in Germany, some of which are named by the Author. For driving these pumps compound rotative engines with fly-wheels are considered the best, Cornish engines having been found to be unsuitable. According to Hoppe's experience, this

kind of pump does not work advantageously when the quantity of water to be raised amounts to less than the fourth part of that for which it is designed.

G. R. B.

Renhaye's Pneumatic Corn Elevator. By A. Marnier.

(Revue industrielle, 1878, p. 201.)

This elevator consists of a receiver placed in the upper part of the grain warehouse; to one side of this is attached a pipe from an exhaust fan capable of supporting a pressure equal to that measured by a column of water 3 inches in height. To the other side of the receiver is attached the pipe which conducts the grain therein. Near the entrance of the grain into the receiver, through this pipe, is a baffle plate placed at an angle of about 45°, which prevents the grain from entering the exhausting pipe attached to the fan by causing it to be projected to the bottom of the receiver. At the lower end of the grain-pipe is a regulator for determining the weight of the semi-fluid column in the pipe according to the pressure therein. This consists chiefly of a piston acted on by air pressure and attached to a sleeve on the lower openwork part of the grain-pipe. The vertical oscillations of this sleeve thus cover or uncover more or less of the air and grain inlet passages according to the pressure in the pipe. The action of the apparatus depends upon the fact that, when pulverulent or finely divided solids are mixed with air in motion in a pipe, they form a semi-fluid, in which the pressure varies as in ordinary fluids. Thus, assuming two points A and B in a vertical semifluid column, A being above B by the distance Z. If p is the pressure of the air at A, P the pressure at B, Q the discharge of solid matter per second, V the velocity of the air, δ the density of air, R the radius of the pipe, g the normal acceleration due to gravity acting on an element of the solid matter, a, the coefficient of friction of the air in the pipe, then—

$$p - \mathrm{P} = \frac{\mathrm{Q\,Z\,V}}{\pi\,\mathrm{R}^2 g} + \frac{2\,a\,\mathrm{Z\,V}^2}{\mathrm{R}} + \mathrm{Z}\,\delta\,\mathrm{V}.$$

In applying this for small pressures, and replacing Z by H for height of elevation, this formula becomes general, and from it is deduced—(1) that in a semifluid column the pressures vary as in an ordinary fluid; (2) that the specific weight of the semifluid column may be augmented to a certain limit; (3) that solids such as grain may be elevated to all heights by regulating the specific weight of the semifluid in accordance with the air pressure obtained; (4) that the maximum efficiency is obtained when the specific weight of the semifluid is at about the maximum. It has been found that the maximum efficiency also depends upon the velocity of the solids on their entry into the

receiver, which should be as nearly as possible zero. To meet this condition, the pipe leading the grain into the receiver is made of increasing size from the foot, the velocity of the grain is thus progressively diminished. In some experiments made with one elevator, the receiver was placed 32·8 feet above the ground, and with a 6-horse engine from 8 to 15 tons of grain were elevated per hour, this elevator not being of the most approved design. Four different forms of the elevator are described, all of which automatically separate the dust from the grain, and one form is described by which the grain is cleaned, separated, and dried by means of hot air. When it is an advantage to break up the grain, it may be sent through the fan and not into the receiver. One of the most important applications of the principle of the design is to portable apparatus for moving ships' cargoes direct into warehouses. For this purpose the receiver is dispensed with, and the fan so arranged as to avoid breaking the grain.

W. W. B.

Oil-Tests. By M. Laugier.

(Bulletin de la Société scientifique industrielle de Marseille, 1877, p. 221.)

The "true industrial value" of a lubricating oil, according to the Author, can only be appreciated by the double process of chemical analysis and mechanical trial. There are two methods of chemical treatment, by the adoption of either of which the proportions of neutral oil and fatty acids is directly determined. The first method depends on the fact that the solubility of fat vegetable oils, excepting castor-oil, in alcohol is nearly nothing, whilst, on the contrary, free fatty acids and glycerine are readily dissolved in it. By lixiviation with alcohol the fatty acids are separated as well as the glycerine, which remain as a sediment, and are weighed, after the alcohol is driven off by evaporation. The second process consists in treating the oil, previously saturated with carbonate of soda, with the aid of ether. Manganate of soda is insoluble in the ether, which only dissolves traces of oleate of soda, whilst it separates the neutral oil. The residue of the evaporation of the ether gives the weight of the neutral oil and of the oleate, which may be separated by washing with distilled water.

The Author describes the oil-test of MM. Deprez and Napoli: an apparatus in which 5 grammes, or 77 grains, of the oil to be tested, are poured upon a polished disc revolving on a neutral axis. Three plates fixed in a stationary disc above the revolving disc, loaded by means of a steelyard to give the pressure required, bear with their edges at an angle of 30° upon the revolving disc, each of them presenting an area of contact of about 1½ square inch. By means of a suitable connection, the stress by friction on the

upper disc is communicated to a pendulum or hanging weight, the angularity of which varies with the quantity of frictional resistance. The deviations of the pendulum are registered on a sheet of paper, for measurement; the ordinates of the diagram being proportional to the total work of friction between the two discs. By a simple inspection of the diagrams produced in this manner, lubricating oils may be classed according to their relative lubricating powers.

D. K. C.

On the Spitzberg Tunnel. By A. STÁNĚ and C. PASCHER.

(Zeitschrift für Bauwesen, vol. xxviii., col. 179.)

The mountain mass known as the Böhmerwald, dividing Bohemia on its south-west side from Bavaria, is so great an obstacle to the passage of railway lines that, notwithstanding the large amount of local business, only a single line, the Bohemian Western railway, had for a long time been made across it. This takes the line of a low pass, about 1,650 feet above the sea-level, at Taus, where the main chain is divided into two parts. A new line of communication between the coal district of Pilsen and Southern Bavaria being desirable, not only as an outlet for coal, but also to develop the timber and glass trades of the district, it became necessary to cross the mountains under less unfavourable conditions; and this has been done by a line going south from Pilsen to the market town of Eisenstein on the frontier, where it will join an extension of the Bavarian railways from Deggendorf. The actual mountain portion of this line, which forms the subject of the present memoir, commences at Neuern, on the Angel, a brook in the Bohemian or Elbe drainage, and by a series of heavy cuttings and embankments, the latter up to 100 feet in height and 270,000 yards cubic contents, reaches the summit in about 15 miles, with a ruling gradient of 1 in 60, and crosses to the Danube drainage by a tunnel of nearly 1,900 yards. An alternative line, on a gradient of 1 in 55, was projected, having a summit tunnel of not much more than half the above length; but as the works in the approaches were of an extremely heavy character, and the line, traversing a length of bare hillside, would have been liable to be blocked with snow for a great part of the winter, the longer line was preferred. The height of the crossing adopted is 2,724 feet above the Adriatic. The line in the tunnel is laid horizontal for 180 yards at the summit, and thence on an incline of 1 in 450 to the northern and 1 in 200 to the southern end. The rocks passed through are crystalline schists, including grey mica schist with quartz, a more contorted and micaceous variety of the same rock, and schists containing graphite, with dolomite, limestone and garnet rock, the whole being traversed by occasional granite veins. No special difficulties appear to have been encountered either from water, loose or exceedingly

hard ground, and only about 42 per cent. of the entire length required to be secured with masonry. The area of the cross section is 47·145 square mètres, or somewhat smaller than that adopted for the tunnels on the Semmering line, but about the same as that of the Mont Cenis, being intended for two lines of rails, though only a single one has been laid at present. Owing to difficulties in obtaining water-power, boring machinery was not used, but two shafts were sunk about 700 yards from either end, 138 yards and 122 yards in depth respectively, and the work was carried out by hand labour, driving from six ends at once. The chief peculiarity seems to have been in the use of small borers, with a mean breadth of cutting edge of 1½ inch, or as narrow as could be obtained consistently with the power of the cast-steel borers to resist crushing under the blows of the 15-lb to 18-lb. mallets used. In sinking the holes were bored about 26 inches, and in driving only 15 inches deep, the strongest dynamite being used for blasting, and fired by safety fuse, the electric method of firing not having given satisfactory results. The sinking of both shafts commenced on the same day, September 22nd, 1874; the deeper one, No. I., was completed in four hundred and nineteen days, on November 22nd, 1875, and the shallower, No. II., in three hundred and fifty-eight days, on September 22nd of the same year, the average rate of sinking being about a yard in three days. The average rate of driving of the preliminary drift on all six ends was 0·608 mètre (2 feet) per day, or 0·676 mètre (2 feet 2½ inches) in the ends driven from the entrances, and 0·548 mètre (1 foot 9½ inches) in the shaft headings. The corresponding quantities for the completed section were 0·674 mètre from the day ends, and 0·485 mètre (1 foot 7 inches) from the shaft headings, or 0·551 mètre (1 foot 9½ inches) for the mean of both. The preliminary drift at the level of the crown of the arch was carried about 43 yards in advance of the first enlargement to the full size of the arch, behind which at a distance of 16 or 17 yards the excavation was increased to about two-thirds of the full section down to the floor level for a length of 20 or 22 yards, leaving a rib of ground on one wall to be cut out by the finishing party. From twelve to thirteen men were employed in each gang of sinkers, making eight-hour shifts. In driving, two pairs of men were employed in the drift headings, twelve to seventeen in the arch, eight to twelve cutting down to the floor, and two to four in finishing the wall. The drift men worked eight-hour and the others twelve-hour shifts. With all points at work from 2,800 to 3,000 drills, weighing about 9 tons, were in use, and the amount of new steel required to supply the waste was about 2 tons monthly. The best Sheffield tool steel, costing on the spot about £3 8*s.* per cwt., was found after numerous trials to be best adapted for the work. From 1,200 to 1,400 drills were sharpened per diem at first, but when a charge of 1 kreuzer per drill was imposed upon the men, the number blunted was reduced to between 700 and 800, without any diminution in working effect. The cubic volume of rock broken in the complete-size tunnel was 90,600 cubic mètres (118,505 cubic

yards), of which 53,600 cubic mètres was trammed out at the ends and 37,000 drawn up the shafts. A 25 HP. steam engine was used for winding, and one of 50 HP. for pumping in each shaft. On the completion of the work, the shafts, not being required for ventilation, were filled up, a certain amount of material being laid in a dry arch over the roof of the tunnel, above which rock was thrown in, the sides being supported internally by arches from wall to wall. This was adopted in preference to merely having an arch near the surface, in order to prevent the possible accumulation of large masses of ice such as would be likely had empty spaces been left. From first to last, about two years and a half were required for completing the work, a speed which the Authors consider to be unmatched in any similar work effected by hand labour. Numerous detailed statements of the results obtained and material consumed in each kind of ground are given in considerable detail, and form a valuable contribution to the knowledge of this kind of work. There is, however, no information as to the cost of the finished tunnel. About three hundred miners, and rather more than the same number of spadesmen and labourers, were employed. These were partly lodged in two barracks on the spot, and partly dispersed through the few farmhouses in the neighbourhood, the latter being the more generally preferred by the men in permanent employment.

H. B.

On the Use of Iron Lining Walls in the Saarbrücken Mines.

(Oesterreichische Zeitschrift für Berg- und Hüttenwesen, vol. xxvi., pp. 249, 261.)

The use of iron for securing the walls of shafts, levels, and other excavations in the Saarbrücken mines has been adopted in special cases for about fifteen years, one of the first instances being a main air-way of the Duttweiler mine, where timbering was replaced by cast-iron door-posts with caps of old mine rails in 1862. The general substitution of round for rectangular shafts in the year 1866 gave the first impulse to the systematic use of wrought-iron linings, while about the same time the peculiar local conditions of the mine of Sulzbach-Altenwald led to a series of experiments on a very large scale on the application of the same material to securing levels. The favourable results obtained in these experiments, together with the fall in price of iron, have been followed by a systematic adoption of the new method to a very considerable extent, and at the same time wrought iron has been in many instances substituted for wood as a material for sleepers in the railways underground.

The first shaft in which iron lining was partially used, apart from those with water-tight cast-metal tubbings, is the No. 3 Dechen pit of the Heinitz mine sunk in 1867. This is a pumping pit 18 feet in diameter, and the object in lining it was not so much to

resist an inward thrust, as to protect the walls from the action of air and water. The lining consists of 2-inch oaken planks hooped with external rings of double angle or T iron at intervals of 39 inches. The flat side of the ring is turned outwards and the stem bedded in the wood, the empty space between the lining and the shaft being afterwards filled up with concrete. In the Richard and Victoria shafts, commenced in 1866, respectively of 12 and 14 feet diameter, the use of iron rings was substituted, in 1868–69, for the walling, adopted in the first 120 yards sunk. The rings are of ⊔ section with the flat side outwards in contact with the wall of the shaft. They are put together in halves with butt-joints secured by fish-plates with four bolts, and are spaced, as in the former instance, 39 inches vertically apart. Every third ring is supported upon cross timbers, the proper distance between the intermediate ones being preserved by struts, which with the planks lining the walls are all of oak. The cross bearers for the shaft guides are bedded into the hollow formed between the flanges of the rings. The adoption of the above system was followed by greatly increased speed in sinking, an advance of 16 mètres (52½ feet) and upwards per month having been attained; as compared with from 5 to 8 mètres, which was the current rate when timbering with subsequent walling was used. As, however, the substitution of dynamite for gunpowder in blasting took place about the same time, the saving cannot be entirely credited to the suppression of masons' work. The saving in expenditure has, however, been very considerable, the iron-lined portion of the Victoria shaft costing £21 10*s.* per yard as compared with £57 per yard in the upper masonry-lined portion.

Numerous other examples are given of the application of the method upon an extended scale, there being now thirteen shafts in the Saarbrücken district in which iron lining is used as soon as the loose surface ground is passed through. The standard system of ring that has been adopted is of ⊔ section, 215 millimètres (8½ inches) on the flat, 87 millimètres (3½ inches) depth of flanges, and 14 millimètres (½ inch) thick. Each ring is made up of four segments united by cast-iron fish-plates, with four bolts to each joint, and weighs, finished, about 16 cwt. Heavy bearing timbers are put in about every fourth or sixth ring. In one of the newest sinkings, the Tränkelbach pits, which are to be carried to a depth of between 700 and 800 yards, the use of timber is almost entirely given up, the cross bearers for the shaft guides being of I and [iron, while the guides are partly of unequal armed I and partly of T iron section. In this case four rings are set at one time working from below downwards, the vertical distance piece being first screwed on to the ring last finished, when the segments of the next ring below are lowered, joined, and screwed into position, and so on until all four are fixed. They are then secured by wedges at the back, the lining planks of oak are fixed, and the empty spaces are packed up with broken rock. Afterwards the cross girders are fitted and screwed into place, and the shaft is

ready for work. All the ironwork receives two coats of red lead, and the fitting of the different parts is very carefully gone over at the surface, in order that the building up in the shaft may be accomplished as quickly as possible.

The latest development of the principle is seen in an upcast shaft of the Camphausen mine. This is 12 feet in diameter, and is lined with rings of a somewhat lighter section than that given above, with a casing of sheet-iron 5 millimètres thick. The struts between the rings are of ⊔ iron, and are placed at eight equidistant points around the circumference, the joints being further stiffened by bracket pieces at the junctions. The lining sheets are 39 inches high so as to fill up the space between two rings, and of a length equal to one-fourth of the circumference. They are made fast, by screws, to the rings as well as to the upright pieces and brackets. In fixing the lining a series of ten rings are placed at a time working from below upwards. The bottom one is supported upon two cross bearers of railway iron, which are removed as soon as the connections with the upper part have been made good.

The second part, p. 261 of the original, deals with the question of iron frames for levels. These, though possessing undoubted advantages in the case of loose or friable ground, were, until the recent fall in the price of iron, too costly to be generally used, being considerably dearer than timber, and in some cases even than masonry. During the past two years of low prices, however, their use has been considerably extended, the requirements of the Saarbrücken mines for 1878 under this head being given as about 2,000 tons of iron.

Four standard systems of frame have been adopted for levels, either of I (unequal-webbed) or ⊔ section, bent with the broad flange or flat side outwards. The first of the above sections is used in two substances, the lighter weighing 11·2 kilogrammes (25 lbs.), and the heavier, 14 kilogrammes (31 lbs.), per mètre. For galleries with double lines of rails, the frames are of T iron, with flat tops and inclined side legs. They are made in two parts united by a fish-joint at the centre of the cap, the weight being 83 kilogrammes (183 lbs.) per frame.

For single-line galleries, similar frames of light I iron, weighing 55 kilogrammes (121 lbs.), are used. Where there is much vertical thrust, the larger levels are secured by closed elliptical rings put together in two parts, the joints being made in the side parts so as to present a complete arch in the roof and floor. The weight of these is 120 kilogrammes (264 lbs.). Lastly, in friable ground, complete circular rings weighing 175 kilogrammes (385 lbs.) each are used; these are also put together in two pieces. In all cases these segments are delivered curved to pattern from the rolling ...ls, and are not painted. They are set in the same manner as ... frames, being placed at intervals of ½ to 1 mètre, according ... nature of the ground, the intervals being packed with laths, ...g-boards, or oaken pickets in the ordinary way.

... underground railways, three different systems of iron sleepers

have been used. The one most in favour is analogous to Vautherin's plan for full-gauge surface lines. It has └─┘ iron cross sleepers at 2 mètres (6½ feet) intervals, with vertical key wedges. Some thousands of yards of lines have been laid in the levels on this pattern, but it is still in the experimental stage, as, although it gives a very durable road, it is almost twice the cost of wooden sleepers. In addition to the above applications, iron guides have been substituted for wooden ones in many of the older shafts, and cast-iron gutters have been placed in the main water levels. These are of semi-elliptical section, open above, and lined with cement, and are found to be more easily cleaned and kept in order than channels cut in the rock.

H. B.

Deep-boring at Goisern. By C. von Balzberg.

(Berg- und hüttenmännisches Jahrbuch der K. K. Bergakademie zu Leoben und Příbram, 1878, p. 231.)

This boring, undertaken with the object of ascertaining the continuance and value of the salt-bearing measures of Aussee and Ischl, was carried down to a depth of 482 mètres (1,581 feet), of which 202 mètres (663 feet) were executed by hand labour. The strata bored through consisted of about 131 feet of morain gravel, between which and the stone head there were 82 feet of clayey sand, flinty gravel, &c.; the solid ground was formed of a succession of different kinds of chalk and shale beds, the former occasionally with flint pebbles imbedded; the last 131 feet were in dark grey dolomite. Commencing with 15½ inches, the diameter of the hole, after the insertion of the last tubing set, was 9½ inches, which dimensions were maintained for the remaining 217 mètres. Four sets of tubing were required, of 15 inches, 13·46 inches, 11·65 inches, and 10·62 inches outside diameter, and which reached respectively the depths of 86, 212, 621, and 836 feet. These sets were of wrought iron, $\frac{3}{16}$ inch in thickness, the seams being made by double rows of riveting to inside strips of iron 3 inches in width, and which, as well as bands at end of tubes, were hammered to a feather edge; each set was provided with a slightly conical steel shoe.

The tools were generally of an ordinary character, the system employed being that of percussion with iron rods. The off-take was sufficient for rods of 56 feet in length; the boring brake (25 feet long) had its fulcrum adjustable so as to give respective proportions between weight and working ends of 1 : 2·75, 1 : 2·62, 1 : 2·25, and 1 : 1·29, and was at first worked by seven to twelve men; on the adoption of steam power it was moved directly by a small piston rod, working in a vertical cylinder of 3 HP.

The main windlass used during hand labour was afterwards replaced by a drum, and this as well as sludger roll worked by an engine of 18 HP.

The boring rods, 56 feet long and weighing 12 lbs. per yard, were screwed and keyed; Fabian's free-falling instrument was used, and could be worked as an ordinary fork piece: the weight piece of 677 lbs. increased in dimensions towards the lower end, and it was found that by so lowering the centre of gravity, the vertical line of the hole could be maintained without using a guide or rosette; it is necessary, however, to take the precaution of reducing the stroke and increasing the number of blows when passing through highly-inclined strata. Of the bits generally employed, those with projecting cutting edges and broad lugs were most effective, except in stiff clayey ground, when blunter edges and smaller lugs were preferred; the enlarging chisel had its blades kept in position by two wires connecting them with higher part of rods, there being intermediate india-rubber rings, and to pass this instrument through the tubes, the blades were blocked against seating with pieces of wood, which after a few strokes were released. A sludger used had its valve placed in a movable seating, which could be raised by a rod from top, thus facilitating emptying of silt.

The time occupied in lowering and withdrawing the rods together amounted to one hour at the commencement of hand boring (20 to 54 mètres); at the depth of 201 mètres, to 4¼ hours. With machine, the same at 201 occupied 2¼ hours, and at 284 mètres, 3¾ hours; the time required for "sludging" was about 2 hours by hand boring, and 1½ hour with machine. The number of men employed averaged eleven during hand boring, and four while machine boring.

The dead weight raised during changing of tools amounted at the depth of 201 mètres to 5,623 lbs., including weight used to counterbalance the chain on windlass side of the pulley, and an allowance of 880 lbs. for friction; the weight raised in striking the blow was 1,320 lbs., which, at the rate of fifteen blows per minute and with 18-inch stroke, required twelve men, equivalent to power required to raise the above dead weight at the rate of 1·68 mètres per minute. The boring was commenced in January, 1872, and finished in March, 1878; an interval of fourteen months appears to have intervened between the periods of hand boring and machine boring, but, with the exception of a broken weight piece, the recovery of which involved a delay of seven months, and a deviation from the vertical line of hole which necessitated the withdrawal of the top set of tubing and the widening of the hole under the second set so as to regain a vertical position for the latter, the work was uninterrupted by any extraordinary accident. The cost of hand boring, after allowing for value of tools, materials, &c., at the completion of the work, came to £7 9*s.* 3*d.* per mètre, and similarly that of machine boring was £9 3*s.* 4*d.*; of these costs the direct wages for boring amounted to £3 9*s.* 8*d.*, and £1 13*s.* respectively. It would appear that the aggregate cost of boring by machine was at the rate of £1 14*s.* 1*d.* more per mètre, but of this excess £1 was an extra cost for recovering the weight piece; and it may be taken

that the average hardness of strata during the machine boring was 50 per cent. greater than in the upper part of the hole, and, in fact, the number of blows which were registered by a counter gave an average of 18,600 and 12,890 per mètre. Again, the boring by machine was at a mean greater depth of 170 mètres.

The analyses of cost during the boring with steam power is as follows:—

	£	s.	d.
1. Cost of general management	0	11	0
2. Actual wages for boring	1	13	0
3. Wood for machine	1	13	0
4. Hospital charges; allowance wood	0	3	8
5. Smithy charges	0	11	0
6. Foreman	0	11	0
7. Wages for finishing tubing sets	0	7	4
8. Forcing down same	0	3	8
9. Fitting machine	0	9	2
10. Materials for smithy, coal, iron, steel, oil and lighting	1	2	0
11. Carriage of machine, &c.	0	12	10
12. Repairing small breakages, &c.	0	5	6
13. Wages for recovery of weight piece	1	0	2
	£9	3	4

The aggregate cost of the boring amounted (exclusive of the last 40 mètres), to £6,027 15*s.* 6*d.*; deducting the value of bore tower tubing, machine and tools, &c., after completion of boring, viz. £2,480, [the actual cost was £3,547 15*s.* 6*d.*, or at the rate of £8 1*s.* 3*d.* per mètre.

Boring with a rope, which was tried on the first employment of steam power, proved a total failure, owing to the incomplete rotation of the free-falling apparatus and uncertain rotation of the ordinary fork piece, and although a special instrument was constructed, it does not appear to have been practically used. The weight piece, which was lost on March 13th, 1877, and recovered in October, became, after some accidents attending the attempts for its recovery, fast wedged in hole along with pieces of tools, and resort was had to dynamite in order to loosen the chisel and to widen the hole. Previous experiences having led to the supposition that the inefficacy of both tarred and tin cartridges at some depth was owing to the conductive property of the wire being affected by the pressure of water, it was tried to explode cartridges by concussion; this also proved ineffectual, although the cap was pierced through, and the dynamite remained dry. By placing a cap in a glass tube, and covering the ends with iron and india-rubber, leaving a small opening in the upper iron plate for a pointed wire to pass through the india-rubber, it was found that the cap detonated; but still, on placing a similarly mounted cap in the cartridge, the latter did not explode. On filling up the dynamite in a gas tube in the same manner, however, an explosion took place, and ultimately hermetically closed glass tubes, of nearly $\frac{1}{2}$ inch thickness, were used and fired by electricity.

It thus appears that, under a pressure of 44 atmospheres, detonating caps and cartridges are useless, if used in the ordinary manner, while the electric spark is unaffected.

The force of the explosion was very local, and notwithstanding that the strata (52 mètres) overlying the explosion were of a loose nature, they were not disturbed; there was no effect upon the tool used excepting a slight splintering of the lower end of the wooden rod, adapted to the free-falling apparatus for giving the blow; and again, when electricity was employed, the only effect was an abrasion of the covering of wire for about 20 inches.

After the withdrawal of the weight piece, which was raised with fishing tongs on being loosened by the above means, the remaining fragments of tools, &c., in the hole (about 80 lbs.) were further reduced by employment of dynamite, and then bored through.

A. S.

A Comparison of the Ordinary and Diamond Methods of Boring.

By L. STRIPPELMANN.

(Zeitschrift des berg- und hüttenmännischen Vereines für Steiermark und Kärnten, vol. x., pp. 3, 121.)

This memoir describes in the fullest detail the results obtained by two borings made for similar purposes, namely, the determination of possible extensions of the coal measures, at Malkowitz, in Bohemia, and at Weyerfeld, near Rheinfelden, in Switzerland, both being undertaken by contractors, the first by Fauck & Co., of Carlsberg, in Austrian Silesia, with rigid rods and free-falling cutting tools, and the second by the Diamond Rock-boring Company, by their well-known rotary tubular cutter, armed with diamond points. The geological conditions were very similar in both cases. At Malkowitz the work was commenced with steam power on the 1st of September, 1875, and continued until June, 1877, when a depth of 1,857 feet (Austrian) had been reached, the average daily progress for 516 working days being 3 feet 7 inches, 22 lines, and the maximum in any one day 16 feet. The diameter at the beginning was 24 inches, and at the bottom 7½ inches, the entire depth being protected by lining tubes of a total length of 3,346 feet; and in addition to the boring proper, 594 feet of ground, fallen in, required re-boring. The first 144 feet passed through consisted of Cretaceous shales, clays, and sandstones; these were succeeded by about 800 feet of red shale and sandstone in about equal proportions, belonging to the Permian series. At 950 feet true coal measures were reached, consisting of grey shale and sandstone, but without workable coal seams, and at 799 feet the transition to the Silurian rocks became apparent. A further depth of 58 feet was bored through, when the evidence of the presence of older slaty rock being undoubted, the work was

stopped at 1,857 feet, the hole being lined with a 7¼ inch tube down to 1,820 feet, and in a condition to be carried to a considerably greater depth if necessary. This line of tubes was recovered, but all those of larger diameter, together about 1,500 feet long, resisted all attempts to move them, and were abandoned. The greater part of the work was done with square iron rods ⅞ inch in the side, and a Fabian's free-falling cutting tool, working percussively; but at different times experiments were made with a hollow rod and continuous flushing current of water, after Fauvelle's manner, both with and without the free-falling cutter, and also with a hollow crown borer, working by rotation, to obtain cores. None of these innovations, however, proved successful.

The total number of 516 working days occupied in boring may be divided over the different operations as follows :—

	Days.		Per cent. of the whole Time.
1. Boring proper	219½	or	42
2. Clearing out sludge	31½	„	6
3. Lifting and lowering rods . . .	81	„	16
4. Enlarging and fixing tubes . . .	61	„	12
5. Repairing accidents	23	„	4
6. Re-boring fallen ground, repairing and cleaning boring gear and engines	101	„	20

The total expenditure, after allowing for machinery and buildings at a depreciation of 40 and 25 per cent. on their first cost, was computed to be £5,040 13*s*., or at the rate of £2 14*s*. 2*d*. per foot, which sum is estimated to leave a profit to the contractor of £1,200 5*s*. The original agreement provided for a new boring being made at the cost of the contractor, in the event of the first being abandoned, on account of any accident whatever, before a depth of 800 feet had been reached. If such an accident had happened, the whole profit would have been absorbed; but, on the other hand, if any workable coal had been discovered a further sum of one-third of the rate agreed upon, amounting in all to about £1,800, would have been paid to the contractors.

The second boring described, that at Weyerfeld, near Rheinfelden, in Switzerland, was undertaken by a local exploring company, with a view of proving a possible extension of the coal measures under the New Red Sandstone into Swiss territory, which it was considered might require a boring 2,500 feet deep. The Diamond boring machine was selected in preference to that of Lippmann & Co., of Paris, on account of its much greater speed, three months being estimated as the time required to bore to the full depth by the former, and three years by the latter. The work was undertaken by the Diamond Rock-boring Company, represented by Herr Schmidtmann, with a specially constructed machine, and operations were commenced on August 14th, 1875, with a 3⅛-inch borer, giving 2-inch cores, a depth of 728 feet being reached on the 1st of September. The falls of ground then

became so considerable, averaging about 130 feet after each withdrawal of the rods, that it became necessary to line the hole. For this purpose the upper part was widened to 7 inches down to 265 feet, and thence to 468 feet to 6 inches, and lined. Below this a 5-inch line of tube was used, which, when difficulties arose with the boring tube, was made to cut its own way by attaching a boring crown with twelve diamonds, and working it by rotation in the same way as the ordinary rods. Between September 22, when the boring was resumed, and September 30th a further depth of 497 feet was gone through, when it became necessary to continue the 5-inch lining, owing to the continued fall of ground. At this depth, 1,225 feet, the bottom of the Permian sandstones were reached, and the borer passed into gneiss rock, which it was at first considered might be only loose blocks, but a further depth of 169 feet bored in the first fifteen days of October showed alternations of crystalline, schists, granite, and diorites, obviously of greater age than the carboniferous series, so that the work was necessarily stopped. The total depth of 1,422 feet was gone through in sixty-three days, and this included not only the operation of boring a 3½-inch hole of the entire depth, but widening at the top 640 feet to 5, 6, and 7 inches, the re-boring and removal of 2,500 feet of ground fallen in, and the fixing of 1,171 feet of lining tubes. Out of these only the 5-inch line, of 777 feet in length, was recovered, those of 6 and 7 inches diameter being immovably fixed, and resisting all efforts made to withdraw them. Apart from accessory operations, the rate of progress for the time actually occupied in boring was 41 feet 9 inches, 10 lines per day of twenty-four hours; but taking these into account, the rate was 22 feet 6 inches, 10 lines. The greatest depth gone through in any one day was 76 feet 8 inches, on the 26th of September, when the boring was 938 feet deep.

The cost of the boring is given in the fullest detail, the total amounting to £7,920, or at the rate of £5 12*s*. per foot.

After describing both methods, the Author analyses the results obtained, and shows that the great speed attained of the diamond borer is in great part to be attributed to the small diameter adopted, and that for equal volumes of rock removed the free-falling cutter is actually quicker in work. The fact that in rocks of greatly similar character the fall of ground in a depth of 1,857 feet was 595 feet, in the Bohemian boring, as compared with 2,500 feet in a depth of 1,422 feet at Rheinfelden, seems to show that in the latter the regular and systematic use of lining tubes was somewhat neglected. The conclusion arrived at is that the greatest advantage of the Diamond boring system will be found at medium depths not exceeding 1,500 feet, but that for very deep borings a system combining the free-falling cutter, with a continuous discharge of the comminuted stuff by a current of water will be found the most advantageous. An appendix describes, with illustrations, a contrivance of this kind combined with a method of obtaining cores, proposed by W. Stoz, of Stuttgart, but not as yet

carried out in practice, and another of a similar character by the contractor for the Malkowitz boring, which was described in the same journal in 1875.

H. B.

On the Use of Dynamite and Electric Ignition in Blasting at Přibram. By J. Hozák.

(Oesterreichische Zeitschrift für Berg- und Hüttenwesen, vol. xxvi., p. 207.)

During the past three years, the use of dynamite instead of ordinary blasting powder has been considerably extended in the Přibram mines; not only in those parts where boring machines are employed, but also for ordinary hand work in shafts and levels, and even in stopping. The most extended application has been in the August-Stefani mine, where the ground is extremely variable in character, being usually hard and tough rock, but full of joints and shakes, with hollow spaces in the vein; all the workings being as a rule rather wet. The result of comparative trials in eight different places, including two sinkings in shafts, two in winzes, and four drivings on levels—which are set out in detail in a table at p. 209—show a saving with dynamite and electric blasting of 23 per cent. in the price per fathom, 28 per cent. in wages paid, and 33 per cent. in time. When safety-fuse and detonating caps were used, the corresponding figures were 9, 11, and 15 per cent. respectively.

These figures therefore show that dynamite, notwithstanding its high price, which is more than double that of powder, is the cheaper explosive of the two in use. The prices charged per kilogramme were 1 florin 62 kreuzers for dynamite, and 74 kreuzers for powders; the detonators for electric firing 4 kreuzers each, and those for ordinary fuze 1 kreuzer, ribbon fuzes for electric blasting in wet ground 5 kreuzers, and patent fuze 37 kreuzers per coil. The latter, a gutta-percha covered fuze, was used for single shots, in both wet and dry ground, ordinary white safety-fuze being found too uncertain for general use.

Frictional electric machines, made by Bornhardt of Brunswick, and costing about £7 10*s*. each, were employed, a total number of six being in use. At first, before the miners were accustomed to handling them, two or three of these were disabled, and required repairs monthly, at a cost of from 8*s*. to 12*s*. Subsequently, this item has considerably diminished, only one machine, as a rule, being sent in for repair per month, and the repairs rarely amount to more than cleaning.

The machine is kept at the surface, until within a quarter of an hour of the time that it is wanted for use underground, when it is sent down in the cage, and carried along the level to the working face. When the shots are fired, it is immediately returned, and

kept in a dry, moderately warm room until the next batch of shots are ready for firing. This continual carrying backwards and forwards, though undoubtedly inconvenient, is absolutely necessary, as, if kept in the mine, the machine soon becomes damp, and will not give sparks.

A certain difficulty was at one time experienced in replacing the fur rubbers of the machine when worn. After several trials, the Author found that pieces of cat or fox fur, slightly covered with a solution of chloride of zinc and dried between boards by three months' exposure to the temperature of an ordinary room, were sufficiently good substitutes for those used by the maker, which, being prepared by a method unknown to the local instrument makers in Bohemia, could not be repaired by them.

H. B.

Forms and Dimensions of Blast Furnaces. By L. Gruner.

(Annales des Mines, 7th series, vol. xii., p. 472.)

Blast furnaces may be divided into three groups according to the forms of their interiors.

In short or squat furnaces (*fours trapus*) the total height does not exceed three times the diameter at the widest part, or $\frac{H}{D}$ is equal to or less than 3. In ordinary furnaces the ratio $\frac{H}{D}$ varies between 3 and 4, and in tall or slender furnaces (*fours élancés*) it is not less than 4, and may even exceed 5.

Of these forms of furnace the *fours élancés* are found to work in every way the most advantageously, producing more metal per twenty-four hours, in proportion to their capacity, than the *fours ordinaires* or the *fours trapus*, and using less fuel.

At Hieflau, in 1855, the alteration of a furnace of 31 cubic mètres capacity, in which the ratio $\frac{H}{D}$ was 4·51, to one of the same height but of 47 cubic mètres capacity, and with $\frac{H}{D} = 3 \cdot 57$, caused at once an increased consumption of fuel.

Up to a capacity of 150 to 200 (5,296 to 7,063 cubic feet) cubic mètres, so long as the slender form of the furnace is retained, each increase in capacity is generally accompanied by a proportional augmentation in the make; beyond this size, the increase of output obtained by a further enlargement becomes comparatively small, and in the very large Cleveland furnaces an increase in capacity of 300 or 400 cubic mètres does not augment the make at all.

The most advantageous dimensions appear to be, in the case of charcoal furnaces, of which the height can scarcely exceed 16 mètres ($52\frac{1}{2}$ feet), a capacity of 60 to 70 (2,118 to 2,472 cubic feet) cubic mètres; and in the case of coke furnaces, where the character of the materials treated admits of a height of 20 mètres, a capacity of 180 to 200 cubic mètres.

The following forms of furnace are recommended as being the most suitable for different classes of materials:—

——	Height.	Diameter at Widest Part.	Total Capacity.	Ratio $\frac{H}{D}$.
	Mètres.	Mètres.	Cubic Mètres.	
1. Charcoal furnace . .	14	3·0	60	4·66
2. Coke furnace, for fusible and readily reduced ores	16	4·0	140	4·00
3. Coke furnace, for less fusible and less readily reduced ores, in large pieces	18	4·5	200	4·00

From easily reduced ores, the first of these furnaces would yield 30 and the second 50 to 60 tons of white iron per twenty-four hours.

In conclusion, the Author points out that in large furnaces, 2·80 to 3 mètres diameter in the throat, the use of a suitable charging apparatus, such as a well-proportioned cup and cone, is indispensable, in order to ensure a proper distribution of the materials.

A table is added of the dimensions and working results of thirty-four furnaces, and the forms of most of these are illustrated in two plates accompanying the Paper.

W. H.

On Cupola Furnaces. By Prof. A. Ledebur.

(Civilingenieur, vol. xxiv., p. 633.)

The chief object of a cupola furnace is the melting of cast iron; any reducing action being of secondary consideration, as only a single chemical operation is intended to take place in the cupola—the partial conversion of the silicon contained in the pig-iron into silica. As a certain percentage of silicon is desirable in cast-iron—which, of all the admixtures of pig-iron, is the most liable to oxidation, when the fluid or semi-fluid iron is exposed to a jet of atmospheric air—it is necessary to select a description of pig-iron containing a surplus of silicon, which is reduced to the right proportion by the oxidizing agency of the blast. No other change

in the composition of pig-iron—especially no reduction—is intended, and this constitutes a fundamental difference between the cupola and the blast furnace.

The blast furnace is used for the purpose of eliminating oxygen —or, as this is mainly achieved by carbonic oxide—of generating this description of gas, and of exposing the ores to its deoxidizing agency. That portion of the blast, which as a matter of necessity is converted into carbonic acid immediately after entering the furnace, has to be reduced to carbonic oxide as quickly as possible, for the purpose of preventing the ores from melting at too early a stage. The fusion of the ores takes place after the reduction has been effected, and is confined to as small a zone of the furnace, and to as short a time, as possible.

The cupola, on the contrary, is solely adapted for melting in the best possible manner, by generating the largest possible number of caloric units, or by converting the whole of the fuel used into carbonic acid by means of the blast.

If a blast is forced into a furnace at a comparatively high pressure, the surface of the jet exposed to the fuel will be comparatively small; the combustion will be incomplete, and carbonic oxide will be the result. If the same quantity of air is introduced into the furnace within the same time, under comparatively low pressure, tho air will expand in the interstices between the fuel as soon as it enters the furnace; the points of contact between the oxygen and the carbon will be multiplied, and a complete combustion, resulting in the production of carbonic acid, will be the result.

The degree of combustion—or the quantity of caloric obtained by it—depends upon the proportion of the surface exposed by the blast and the fuel to each other. This offers an explanation of the fact, that a dense fuel has proved to be advantageous in the cupola, where the generation of caloric is the principal object aimed at. Charcoal will never give as good results as coke, because the surface offered by it to the blast is far too large, and could only be made proportionate by a partial evacuation of the interior of the cupola. Other facts are, partially at least, explained by these considerations, viz., that the percentage of caloric units actually utilized is, on the average, higher in charcoal than in coke blast furnaces; that it is higher for soft charcoal than for the harder descriptions; that taking off the waste gases by suction is injurious in the case of a blast furnace, as by a diminution of the pressure (or by an increase of the surface of the blast) the fire is drawn upwards in the furnace, while in the cupola it could only act beneficially.

A partial reduction of the carbonic acid, produced by the blast of a cupola, to carbonic oxide cannot be prevented. The additional quantity of carbon effecting this, and the necessary caloric for converting this carbon into a gaseous state, constitute a loss, which, however, is diminished by the use of a dense fuel, and by the cir-

cumstance, that the pig-iron while melting absorbs a considerable quantity of heat from a portion of the carbonic acid generated by the blast, whereby the temperature of this portion is lowered to such a degree that its reduction to carbonic oxide becomes impossible. This process may be daily noticed in any foundry. Before turning on the blast, about half the cupola has been filled with coke; the consequence is, that more heat is generated than can be absorbed by the pig-iron; carbonic oxide is profusely generated, as shown by the large blue flame that escapes. This flame, however, gradually decreases, and, under good management, disappears altogether. As soon as the blowing out begins, and the charging of pig-iron ceases, the flame reappears with a yellow tinge, due to the particles of incandescent carbon thrown up by the blast.

The second object aimed at is the complete utilisation of the heat generated; in this case losses are principally occasioned by the heat that escapes with the waste gases, and by conduction of the lining and casing.

The first of these losses may be considerably reduced by a proper height of the cupola—from 8 to 10 feet above the tuyeres being sufficient to reduce the temperature of the waste gases to an average of 120° Fahr. A greater height hardly proves advantageous in this direction, while it facilitates the generation of carbonic oxide and offers resistance to the blast.

The quantity of heat lost by conduction cannot be lessened by an extra thickness of the lining, as was formerly believed, but may be very materially reduced by melting the pig-iron down quickly. The ratio of the number of heat units produced by the generation of carbonic acid to that by carbonic oxide, is as 8 to 5 for equal quantities of atmospheric air, and as 3 to 1 for equal quantities of carbon; the quantity of pig-iron melted will, as a matter of course, increase, and the loss of heat per unit of weight of pig-iron occasioned by conduction will decrease in the same ratio.

All modern constructions of cupolas which have been attended with more or less success, have been based upon these two conditions—the greatest distribution of the blast, for the purpose of bringing down the pressure and enlarging the surface of contact between the blast and the fuel—and plenty of it, in order to increase the quantity of pig-iron that is to be melted down within a certain time. For the case of Ireland's construction this is effected by two rows of tuyere holes, which are distributed around the normal section of the cupola, the cross sections above and below the tuyeres being enlarged, with a view to produce a slow rise of the gases generated by combustion, for depriving them of their heat, and to provide a sufficient cubical capacity for an accumulation of molten iron. For Krigar's cupola the blast is admitted through two large opposite apertures, by which a most ample distribution of it is effected. McKenzie's cupola is provided with a number of oblong tuyere holes, their longitudinal direction being parallel to the axis of the shaft. If these systems are carefully

carried out, the results will be almost identical—the ratio of the weight of pig-iron to that of the coke, by which it is melted, being a maximum of 100 to 6. As for durability and absence of waste of iron by oxidation, Krigar's cupola has never yet been surpassed.

The following data have been proved by practical experience:—

The pressure of the blast should only be due to the obstruction offered by the coke and iron in the cupola, and never to a diminished section of the tuyeres; it ought not to be less than 8 inches of water column (0·28 lbs. per square inch), and not much more than double that.

The aggregate sectional area of the tuyeres should not be less than $\frac{1}{8}$ of that of the shaft; it is frequently as $\frac{1}{2}$, and even more.

The sectional area of the narrowest portion of a cupola-shaft should be about 150 to 190 square inches for each ton of pig-iron to be melted per hour.

The quantity of blast per second required to melt a ton of pig-iron per hour is from 41 to 49 cubic feet per square foot of the sectional area of the shaft.

The fore-hearths introduced by Krigar answer their purpose of accumulating a larger quantity of molten iron well; and for this reason they are principally suitable for Bessemer works. They afford another important advantage—rendering the melting process independent of the varying level of the fluid iron in the cupola, and both iron and slag are tapped from them more easily than from the cupola itself. Instead of the square fore-hearths, as made by Krigar, a horizontal section of a circular shape is preferable on account of its greater cheapness and smaller exterior surface. Another peculiarity of Krigar's cupola—the door by which the bottom is closed up—is of advantage whenever it is required to drop the contents of the cupola into a wagon for removal from the foundry.

The difference of opinion formerly existing as to the thickness of the lining has been settled by experience in favour of thin linings; generally speaking, a thickness of 7 inches is sufficient for a cupola working 3–4 hours a day; in case of a longer working time, the thickness should be increased to about 10 inches, and for cupolas working all the day long it is taken 11–12 inches thick. Thick linings absorb more heat, which is wasted every time the cupola is blown out; besides, they are not cooled by the atmosphere as effectively as thin ones. Cooling by water (as in the case of blast furnaces) might prove advantageous for the intensive working of Bessemer cupolas. At all events it is judicious to let the upper portion of the lining rest upon a cast-iron ring, bolted to the casing, in order to allow the lower portions to be removed and renewed independently.

The chimney of a cupola should be built on a cast-iron frame, supported by columns, so as to be entirely independent of the furnaces. The expansion and contraction of the latter does not injure the chimney, which in addition is cooled more efficiently by a draught of air during blowing out.

A. H.

Perrett's Furnace for Burning Pulverulent Fuels.

(Bulletin de la Société d'Encouragement, 1878, p. 371.)

This furnace, invented by M. Michel Perrett, consists of a cubical chamber containing a series of firebrick shelves, one above another, upon which the fuel is spread, and is gradually consumed. The chamber is 1 mètre wide, 1·85 mètre long, and 1¼ mètre high; or 3·28 feet by 6 feet by 4·10 feet. There are four shelves, each 1½ mètre, or 5 feet long, being 1 foot shorter than the furnace-chamber. They reach from the front and the back alternately, and the fuel may be lowered from one to another successively. The furnace is in the first place raised to a red heat, by burning wood on all the shelves, as well as in the ashpit at the bottom; then all the shelves receive a first charge of the fuel. At the proper time, the fuel is lowered by means of suitable fire-irons, introduced through doorways in the front of the furnace, from each shelf to the shelf below it, on which it is evenly spread; and the uppermost shelf, now empty, receives a fresh charge of fuel. This process of arranging and charging the furnace is repeated at intervals of twenty-four, twelve, or six hours, according to the strength of the heat required. The rate of combustion is as follows:—

Interval.	Fuel per Square Mètre per Hour.	Per Square Foot.
24 hours.	2 kilogrammes.	0·41 lb.
12 "	4 "	0·82 "
6 "	8 "	1·64 "

These quantities are taken as for pure fuel, ashes excluded. The quantity of air admitted is regulated to a high degree of exactness. It is heated—to 570° F., it may be—and is admitted exclusively at the entrance to the ashpit.

The Perrett furnace is successfully at work in many places: for evaporating liquids, for stoves of all kinds, and for heating apparatus.

D. K. C.

On the Working of Regenerative Puddling Furnaces.

By D. Borbély.

(Zeitschrift des berg- und hüttenmännischen Vereines für Steiermark und Kärnten, vol. x., p. 209.)

The Author gives the result of his experience of the working of the regenerative puddling furnaces under his charge at the Salgó-Tarján works in Upper Hungary, where lignite is used as fuel. The ordinary stack draught puddling furnace of the same neighbourhood is given as a basis of comparison. This turns out yearly about 600 (metrical) tons of puddled bars with a consumption of

about 112 per cent. of pig-iron, and 250 to 300 per cent. of coal. Although the pig metal treated is of a good class, the product is not of satisfactory quality, as the temperature attained is not sufficiently high to carry out the process properly, and from the small production the cost of labour is proportionately increased. The cost of construction and maintenance on the contrary are but small, the latter probably not exceeding *5d.* to *7d.* per ton. The management is also easy, and requires no very close supervision. The metal treated is usually grey or mottled, charcoal pig, white and crystalline kinds being rarely used.

The regenerative furnaces, according to present experience, turn out about 1,800 tons of puddled bars annually, but are probably still below their maximum productive power, or are more than equal to three ordinary furnaces. This production does not, however, require an excessive amount of labour, as the number of hands employed in twenty-four hours is 10 furnace men, and 3½ firemen for the gas producers and boilers, or together 13½ men, while the corresponding number for three ordinary furnaces is 18. The actual saving in wages is about 10 per cent. The loss in working is considerably less than with the ordinary furnace, 100 of puddled bars requiring only 102 to 103 of pig-iron and 240 of coal, the latter including that required for boiler firing. The large yield is not, however, due to heavy fettling, nothing being used for the purpose beyond the cinder and scale produced in the works.[1]

The cost of maintenance is, however, considerably higher than with the common furnace, amounting, according to the detailed accounts given, to *1s.* and *1s. 4d.* per ton of puddled bars. This the Author considers to be due in part to defective construction, and that by judicious modifications in the structure this item may be reduced to an equality with that of the ordinary furnace.

There is, however, an essential difference between ordinary and gas furnaces which tends to restrict the use of the latter to special classes of iron. Speaking generally, a strongly oxidizing body of flame—*i.e.*, one containing a large excess of unconsumed air—is required during most of the stages of the puddling process. According to Dr. J. Kollmann the requirements in free oxygen are about 8·3 cubic mètres per 100 kilogrammes of pig-iron treated. Unless this condition be fulfilled, the refining action of the slag ceases, from the formation of thinly-fluid acid silicate; and oxidation is retarded or stopped, so that the furnace works cold, producing an inferior quality of iron, at the same time the sides and bottom of the hearth, being no longer protected by a nearly infusible coating of basic slag, become exposed to the destructive action of the flame. With the ordinary grate-fire furnace there is no difficulty in varying the quantity of air at any moment without influencing the fire; and indeed for the most advantageous combustion of the

[1] Probably if a large amount of fettling were used the inconveniences subsequently noticed might be remedied.—H. B.

fuel, an amount of air double that theoretically required, is practically necessary. With the Siemens furnace, however, a large excess of unaltered air acts prejudicially, not only by cooling the regenerators when continued for any length of time, but by shortening the flame to such an extent as to prevent the whole surface of the bed from being uniformly heated. This leads the Author to the conclusion that the gas furnace is better suited for the treatment of white iron, poor in carbon and as free as possible from silicon, and that the greyer and siliciferous kinds should be left to the ordinary furnace. The Paper concludes with the working sheets of two furnaces for eighteen months, which give the average number of heats as from 6 to 7 per shift with 12-cwt. charges; the loss on the metal charged as 29 per cent., and the consumption of coal as 152 per cent. in the gas producers, and 88 per cent. in the boilers. There are also specifications with prices of the gas furnaces and producers, the former costing £797 2*s*., and the latter £130 18*s*. each; but as 5 producers are required for the supply of two furnaces, the cost of a single furnace in working order may be put at £1,125.

H. B.

On the Classification of Iron and Steel. By Dr. Rohrig.

(Organ für die Fortschritte des Eisenbahnwesens, vol. xv., p. 175.)

The Author objects to the classification proposed by the committee of the German Railway Union,[1] on the ground that it is of a one-sided character, and that the experiments (chiefly of Bauschinger and Jenny) on which it is founded were not sufficient to afford a trustworthy basis for it. These experiments have been confined to measuring the absolute strength and ultimate reduction of area under steady loads, without taking any account of sudden shocks. But in most of their applications (e.g., rails, rolling stock, &c.) iron and steel suffer more from shocks great or small (the latter being called vibrations) than from steady loads; and their strength under such circumstances should be inquired into. Some writers (e.g., Knut Styffe and Redtenbacher) have deduced the behaviour under impact from that observed under steady loads; but this is a very uncertain method, especially with elastic materials, such as soft iron. It is essential, in considering resistance to impact, that the elastic limit of the material and its laws should be taken into account, as well as the ultimate strength. The Author quotes Fairbairn on "The Effects of Time on Wrought-iron Girders" to prove that, whenever a piece is strained by a blow or otherwise beyond its elastic limit, its resistance against the next application of strain is weakened; and hence the elastic limit is

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. lii., p. 359.

that which really determines the power of the material to resist successive shocks. Thus glass has great strength against a steady strain, but a low elastic limit, and is therefore easily broken by a shock. The proportion which exists between the amounts of *vis viva* required for rupture in the case of sudden shocks and steady strains is stated by Kirkaldy, as the result of his experiments, to be 80 per cent.; but Styffe disputes this, and the point should be cleared up by direct experiment.

The Author proceeds to quote a minute of the Austrian Mines and Ironworks Union, which states that, having fully considered the advisability of a classification of iron and steel, they have decided against the attempt, on the ground of its liability to be upset by further discoveries and researches, and the probability of its leading to dangerous consequences in relation to manufacture. They also consider that the principles of the classification proposed by the German Railway Union are of too partial a character, and would exclude unjustly the iron and steel of certain special districts.

W. R. B.

The Mines and Works of Almaden. By H. Kuss.

(Annales des Mines, 7th series, vol. xiii., p. 39.)

The deposit worked at Almaden consists of three parallel and nearly vertical beds of grit or quartzite, impregnated with cinnabar. The length of the workable portions of these is 150 to 180 mètres, and the thickness of each is from 3 to 8 mètres. Their direction is approximately east to west. The most southern is termed the vein San Pedro y San Diego, and the two others, which lie close together, are known as San Francisco and San Nicolas. The vein San Pedro y San Diego consists of a white grit, regularly impregnated with cinnabar, which gives it a beautiful vermilion colour, particularly towards the western end, where it is richest. Towards the west, the deposit ends abruptly against a mass of schist. Towards the east, it becomes gradually poorer, and passes insensibly into ordinary white quartzite. The eastern end also becomes poorer in going down; so that the rich portion forms, in the deposit itself, a column dipping to the west. The grit forming the veins San Francisco and San Nicolas is black, and harder, more compact, and less regularly and less richly impregnated with cinnabar than that of the other. The beds of grit rich in cinnabar are contained between other barren beds, in some places of schist, and in others of quartzite.

The mine is worked by ten levels, of which the lowest is about 289 mètres below the mouth of the San Teodoro shaft. The first four levels, down to 140 mètres from the mouth of the shaft, are ruinous and inaccessible.

In the lower levels the deposits become more extensive, thicker,

richer, and more regular. The veins San Francisco and San Nicolas approach each other in going down, and in parts join into one at the 9th level, the lowest at which they have been worked; and both approach nearer to the vein San Pedro y San Diego, so that it seems likely that at a still greater depth they will all join into one mass.

At the fifth level (170 mètres), which has now been long worked out, the ore was of very inferior quality. At the sixth it was chiefly "poor ore," containing 1 to 7 or 8 per cent. of mercury, with a few masses of "medium ore," containing 8 to 20 per cent., at the ends of the veins San Francisco and San Nicolas. At the seventh level "rich ore" appeared, containing over 20 per cent., and in some instances as much as 80 or 85 per cent. of mercury; and the proportion of this increases in going lower, until the tenth level, so far as it has been opened, yields nothing else.

When medium or rich ore is calcined, so as to expel the sulphur and mercury, the siliceous residue left is porous and friable, or even crumbles into sand. A portion of the substance of the rock seems thus to have absolutely disappeared, and to have been replaced by cinnabar. The period at which the cinnabar was thus introduced cannot be determined with any approach to certainty. Its introduction was evidently not contemporary with the deposition of the silurian and devonian beds in which it occurs, and dates probably, like that of the cinnabar found in the Palatinate, and at Vallata, near Agordo, from the close of the Permian epoch. It is thus distinctly older than the Idrian deposit of cinnabar, which is regarded as belonging to the triassic period.

Working of the Almaden Mine.

The workings of the Almaden mine communicate with the surface by three shafts, of which one is sunk to the depth of the tenth level, and the others to a little below the ninth.

The upper levels are at irregular distances apart; but in those recently driven, the depth from one to the next is fixed at 25 mètres.

The method of working now in use was adopted about the year 1804. Its essential feature is the use of cross arches and walls of massive masonry, to support the sides of the excavations when the ore has been removed.

Where the veins San Francisco and San Nicolas come close together, the ground between them is entirely removed, and the main arches are turned from the south wall of the one to the north wall of the other.

The work underground is nearly all done by contract. Between 700 and 800 men are employed in the mine in the course of each day, in six-hour shifts, but the whole number engaged is much greater than this—from 2,250 to 2,500, as the men do not, on the average, work more than one shift of six hours every three days, and those occupied in the work that is most injurious to health—

the miners breaking out the ore, and the masons engaged in underground walling—do not work more than one shift in five or six days. The rates paid give the miners 4 to 5 francs for each shift worked.

Until 1873, the raising of the ore in the shafts was done wholly by horse-power, the only engine in use having been an old pumping engine, by Watt, erected in 1791. There is now a good winding engine at each of the three shafts, two being used for ore and materials, and the third for sending up and down the men.

The amount of pumping required is very limited, the quantity of water raised being only from 72 to 84 cubic mètres (15,840 to 18,480 gallons) per twenty-four hours. It is lifted from the seventh level by the winding engine of the principal shaft, and so much as comes in below this level is pumped up to it by hand.

The ventilation is effected by natural circulation, aided, especially in summer, by a Guibal fan at the top of the upcast shaft.

The ore raised contains on the average 7·5 to 9 per cent. of mercury. It is sorted by hand into three classes:—

Metal containing from . . .	21·5 to 25	per cent.
China „ „ . . .	6 „ 7·5	„
Solera „ „ . . .	0·3 „ 0·8	„

Two forms of furnace are in use; the Bustamante furnaces, introduced in 1633, and so-called Idrian furnaces, adopted about the commencement of the present century.

There are twenty Bustamante furnaces, arranged in pairs. The furnace proper of each consists of a vertical cylinder of masonry, 2 mètres in inside diameter and 6·50 mètres high, which is fitted with charging openings in the side and at the top, and divided about the middle of its height by an arch of perforated brickwork. The upper part receives the charge of ore, and the lower is the fireplace. The fuel used is brushwood. A chimney communicating directly with the fireplace promotes the draught, and carries off the greater part of the smoke.

Openings lead from the upper part of the furnace to twelve parallel rows of earthenware condensers or aludels. Each of these is in the form of a vase, open at top and bottom, and they are inserted one in the next, to the number of forty-five or fifty in a row, and well luted at the joints, forming twelve small flues, of variable section and with thin walls. They open at the end into low chimneys, two to each furnace, fitted with dampers for the regulation of the draught. The charge of ore is about 11½ tons. Broken stone or poor solera ore is filled in, first, on the perforated arch, to a thickness of about 16 inches, and the richer ore, first china and then metal, is charged upon this. The bacisco is put in last, and the charging openings are then closely luted up. A fire is next lighted below, and is kept up for eight or ten hours, consuming 2·2 to 2·5 tons of wood. The ore is then sufficiently kindled, and the fire is let out; but the mass of ore is maintained at a red heat by the combustion of the sulphur that it contains,

and continues to calcine spontaneously, until this and the mercury combined with it have been expelled. The residue is then allowed to cool, and is discharged, and a fresh charge is put in. Each operation lasts seventy-two to seventy-five hours, thus divided: charging, one hour; firing, eight to ten hours; calcination, forty-five to forty-six hours; cooling, eighteen hours.

The mercury condenses chiefly in the portion of each row of aludels nearest to the furnace, and flows from the aludels through openings 2 to 4 millimètres in diameter in the under side of each, into suitably arranged gutters, and thence into a reservoir. The aludels nearer to the furnace are taken up at intervals of fifteen days, and those more distant every two months, to clear out the mercurial dust or "soot" that gathers in them.

The Idrian furnaces, of which there are only two at Almaden, differ from the Bustamante furnaces in little except their greater size, and the form and arrangement of the condensers. The furnace is 3 mètres in inside diameter by 7·50 mètres high, and the fireplace is separated, as in the Bustamante furnace, from the chamber that receives the charge by an arch of perforated brick-work. Each furnace communicates with twelve masonry condensing chambers arranged in two series of six. The condensers are lined with Portland cement, and the mercury from them is led by pipes into a stone reservoir. The charge is between 28 and 29 tons, and each operation lasts six days: one for cleaning and charging, one for firing, two for calcination, one for cooling, and one for discharging.

The mercury is led from each furnace to the magazine by a wrought-iron pipe, and is there put up for sale in wrought-iron bottles containing each 34·507 kilogrammes, or 75 Castilian pounds of the metal.

The cost of each operation in the Bustamante furnace is about 50 francs, equivalent to 4·50 francs per ton of ore treated, or to 60 francs per ton of mercury. The cost in the Idrian furnace is about the same, 125·50 francs for each operation, or 4·33 francs per ton of ore.

The loss of mercury in the process of distillation has been determined with great care, and does not exceed 5 per cent. of the quantity contained in the ore treated, in the case of the Bustamante furnace, or 5·5 to 6 per cent. in that of the Idrian furnace.

The cost of getting and raising the ore, which yields on the average 7 to 7·5 per cent., is about 55 francs per ton, and the total cost per ton of the mercury, bottled ready for sale, is as follows:—

	Francs.		Francs.
General charges	160	to	200
Ore	700	„	900
Distillation	45	„	60
Putting up in bottles	170	„	180
Sundries	50	„	100
	1,125	„	1,440

Or from 38·80 to 49·70 francs per bottle.

The selling price, in London, is very variable; the average, in the twelve years from 1865 to 1876, was £10 9*s.* 5*d.* per bottle. The annual output is about 35,000 bottles, and may be easily increased, in the course of one or two years, to 40,000 bottles, or 1,380 tons, at a cost of less than 42·50 francs per bottle.

The mercurial emanations in the mine and in the distillation works are very injurious to the workmen, none of whom, if they have been long engaged at Almaden, escape salivation and ulcers in the mouth; and many suffer from a kind of palsy, accompanied by an almost total loss of strength, and a sad weakening of the intellect.

A short sketch of the history of the mine concludes the Paper.

W. H.

The Mineral Produce of the Prussian States in the year 1877.[1]

(Zeitschrift für das Berg-, Hütten- und Salinen-Wesen, vol. xxvi., p. 7.)

These statistics appear in a slightly altered form this year, consequent upon alterations in the arrangement of the originals. The new terms "melted" and "welded iron" are introduced for the first time. The former includes all cast steel, and the latter, malleable iron, puddled and crude steel.

Coal and Iron Ore Mining.—The number of collieries at work during the year was four hundred and twelve, besides fifteen new winnings and suspended workings. Of lignite mines, five hundred and nine were in work, and sixteen others were either in course of development or stopped. Iron ore was raised in five hundred and eighty-one mines, ninety-six others being either stopped or not raising ore. The production of these three staples was as follows:—

Mineral.	Quantity.	Colliery Consumption.	Number of Workpeople.			
			Under-ground.	Surface.		Totals.
	Metric Tons.	Metric Tons.		Men.	Women.	
Coal	33,672,026	2,428,4[illegible]7	117,650	26,097	2,168	145,915
Lignite . . .	8,636,598	728,738	10,673	7,803	265	18,741
Iron ore . . .	2,752,486	..	18,272	5,297	1,450	20,019

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. l., p. 282, for similar statistics for 1876.

The value of these minerals at the place of production was:

		s.	d.		s.	d.		s.	d.	
Coal .	Maximum	10	2½	Minimum	4	9½	Average	5	7¼	per ton.
Lignite	„	12	7¼	„	2	7¼	„	3	4¾	„
Iron ore	„	12	0	„	2	2½	„	6	2½	„

The production of the more important metallic ores, other than those of iron, was:

Mineral.	Number of Mines.	Quantity.	Average Value per Ton.			Hands employed.			
						Under-ground.	Surface.		Totals.
							Men.	Women.	
		Tons.	£.	s.	d.				
Zinc	113	575,147	0	19	2½	7,240	2,847	2,255	12,342
Lead	146	134,582	8	4	0	9,471	8,632	491	18,594
Copper . . .	96	336,447	1	2	0	5,996	1,228	8	7,227
Silver and gold (cwt.)	..	91	1,115	1	7	Included under lead.			
Nickel. . . .	3	222	3	15	2½	15	8	..	23
Cobalt. . . .	1	79	15	0	0	50	10	..	60
Arsenic . . .	4	631	1	1	5	37	29	..	66
Manganese . .	34	5,289	1	16	9½	149	109	32	290
Iron pyrites . .	10	67,879	0	16	5	372	157	7	536

The total produce of metallic ores of all kinds was 3,942,340 tons, valued at £2,971,109, or an average of 15*s.* per ton.

Rock salt, and other mineral salts—kainite, boracite, &c.—were raised in nine mines, employing twelve hundred and twenty-two hands, to the extent of 374,144 tons, valued at £130,781. The salt produced from brine works amounted to 227,561 tons, worth £280,250, exclusive of the excise duty. Chloride of potassium was produced from the Stassfurt salts in eleven refineries, whose total make was 44,537 tons, valued at £257,842. Each ton represents the result of the treatment of between 7½ and 8 tons of the crude salt.

The number of fatal accidents in mines of every description was five hundred and thirty-four, out of a total of 231,117 persons employed, or 2·310 per thousand. In 1876 the proportion was 2·491 per thousand, on a total number employed of 240,865. The number and proportion of accidents in the different classes of mines were:

In coal mines .	496,	being 1 per	362 hands,	or 1 per	83,145	tons raised.	
In lignite mines	33	„ 1 „	570 „	„ 1 „	261,715	„	„
In metal mines.	76	„ 1 „	747 „	„ 1 „	44,270	„	„
In other mines.	19	„ 1 „	450 „		..		

As compared with the preceding year, the proportion of fatal accidents in coal mines was about 3½ per cent. less; in lignite mines, about 40 per cent. less; in metal mines, about 25 per cent. less, and in other mines, which include mineral works of all kinds not otherwise classified, about 80 per cent. more.

The principal causes of accidents in coal mines, and the loss of life chargeable to each, were:

1. Fall of ground	156
2. Explosions of fire-damp	22
3. Accidents in blasting	37
4. Accidents in shafts and inclines . . .	136

Metallurgical Produce. Iron.—Of a total number of two hundred and thirty-four blast furnaces smelting iron ores, one hundred and ninety-two were in blast for an average period of not quite eight and a half months each. The iron producing materials and fluxes melted were:

	Tons.
Iron ores, home produce	3,176,347
" " foreign	250,643
Forge and mill cinders	236,931
Scrap and washed iron	5,246
Burnt pyrites residues	5,343
	3,674,510
Limestone	1,145,636

467,140 tons of iron ore, or about 14 per cent. of the whole amount, were used in the calcined state.

The total make of pig iron was 1,421,667 tons, equal to an average of 201 tons per furnace per week, as compared with 178 tons per week in 1876. According to fuel employed, the blast furnaces were classified as follows:—

Fuel.	Number of Works.	Number of Furnaces Blowing.	Foundry Pigs and Castings.	Steel Pigs.	Forge Pigs.	Average Weekly Make.
Coal and coke	63	144	72,848	434,136	854,656	256¾
Charcoal . .	42	42	27,767	635	6,502	28¼
Mixed . . .	6	6	2,930	..	12,264	61½

The corresponding weekly averages in 1876 were: coal and coke furnaces, 232 tons; charcoal furnaces, 27⅓ tons; mixed fuel furnaces, 51⅛ tons.

The total number of hands employed in the production of pig iron and first-fusion castings was 13,801, including 518 women, or

less by 936 than the number returned as similarly employed in the preceding year.[1]

The consumption of imported pig iron, and the different purposes to which it was applied, appears to be as follows. (The table given in the previous years is not continued in the present form of the statistics.)

	Tons.
Foundry use	166,500
Malleable iron	32,067
Steel making	165,623
	369,190

Welded Iron, Malleable Iron, and Forge Steel.—The total number of works producing this class of iron was 289, employing 36,386 hands, and containing the following furnaces:

Finery fires (bloom forges)	186,	of which	187	were in use.
Puddling furnaces	2,060	"	1,240	"
Rotatory "	2	"	none	"
Heating "	1,059	"	622	"
Annealing "	374	"	273	"
Cementation steel converters	8	"	2	"
Other kinds[2]	208	"	105	"

The materials consumed were:

	Tons.
Pig iron	1,035,183
Purchased blooms and puddled bars	34,106
Scrap and waste iron	87,243

The total welded products were, 946,455 tons of iron, and 254 tons of blister steel.

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[1] By comparison with the preceding year it would appear that one hundred and five furnaces have been pulled down or abandoned during the year, as seen in the following figures:—

——	In.	Out.	Total.
There were in 1876—			
Coal and coke	115	119	234
Charcoal	50	43	93
Mixed fuel	7	5	12
			339
Reduced in 1877 to—			
Coal and coke	144	31	175
Charcoal	42	10	52
Mixed fuel	6	1	7
			234

This accounts for the sudden increase in the average makes, the oldest and less effective having been removed.

[2] Include various mill furnaces, small hammer forges, and smiths' fires.

Molten Iron (Cast-steel) Works.—There were forty-two in number, employing 17,319 hands, and containing the following furnaces:—

Bessemer converters	57, of which	24 were in use.	
Open hearth smelting furnaces	36	" 10	"
Crucible furnaces for the production of steel	11	" 8	"
Crucible furnaces for steel melting	315	" 93	"

The above are those principally concerned in production, as auxiliary furnaces are returned.

Cupolas	83, of which	32 were in use.	
Reverbatory smelting furnaces	27	" 3	"
Heating furnaces	37	" 7	"
Reheating and annealing	507	" 230	"
Other kinds[1]	63	" 17	"

The materials worked up included—

	Tons.	
Pig iron	410,074	Employed in the production of flusseisen, i.e., chiefly Bessemer and Siemens steel.
Spiegel	34,683	
Ferromanganese	1,591	
Malleable iron	2,200	
Steel blooms	1,250	
Old iron and scrap	32,959	
Steel	8,836	Used in the production of crucible steel.
Forge steel	35	
Malleable iron	3,093	

The quantities of the different classes of steel produced were—

	Tons.
Bessemer	396,519
Siemens	42,564
Other kinds	120

Of the above quantities, 5,315 tons were converted by subsequent fusion into crucible steel, in addition to 27,199 tons made from malleable iron and other materials in the crucible furnaces direct.

The joint total of melted iron and crucible cast steel is returned at 461,093 tons, and the value as £2,866,805.

The production of finished iron and steel, classified according to uses, was as follows:—

——	Iron.	Steel.	Together.
	Tons.		
Rails and fish-plates	41,753	297,201	338,954
Sleepers	35,921	..	35,921
Railway wheels, tires, and axles	15,431	44,373	59,804
Bridge and girder iron	78,293	111	78,404
Heavy plates and forgings	72,005	12,388	84,393
Artillery and projectiles	..	16,852	16,852
Black plates and sheets	64,577	9,233	73,810
Tin plates	7,633	..	7,633
Wire	137,911	173	138,084
Tubes	4,068	..	4,068
Other kinds	358,795	13,939	372,734

[1] ...uding drying, tempering, clay burning, spiegel melting, and other furnaces.

The other metallurgical products included—

Metal.	Number of Works.	Hands.	——	Value.
				£
Zinc.	35	6,443	94,744 tons	1,685,885
Lead and litharge.	27	2,870	80,887 „	1,613,407
Copper	19	1,628	8,193 „	624,536
„ unrefined and regulus .	19	1,628	248 „	5,100
Silver	21	..	223,147 lbs.	899,855
Gold	7	..	310·14 „	21,534
Nickel	4	158	151,400 „	27,265
Cobalt products	1	..	22,000 „	19,000
Arsenical „	4	11	110¼ tons	1,592
Cadmium	6	..	4,043 lbs.	1,210
Antimony from metal dross . .	1	20	23¼ tons	2,453
Sulphur	6	..	1,198 „	9,543

H. B.

Warming and Ventilation of Buildings. By E. Trelat.

(Résumé de la Société des Ingénieurs civils, 1878, p. 85.)

The Author gives a brief outline of the several methods of warming and ventilation in France, and remarks that forty years ago no combined apparatus was in use. Even now, with all the improvements in the various methods followed, numerous complaints are made, especially in regard to the quality of the air supplied; and regret is often expressed that the old-fashioned large open fireplaces, with all their evils, have been abandoned.

The cause of these complaints is that, in nine cases out of ten, the improvements in the apparatus have been in the mode of generating heat, in its transmission, with increased purity of the air supplied, all of which improvements do not touch upon the vital point. The conditions that would involve an increased or decreased supply of air for ventilation bear no relative proportion to its required temperature, or, in other words, the Author believes that warming and ventilation must be distinct operations, and of independent action. Amongst the general principles governing good ventilation, the supply of air to any apparatus should be taken from a high level, with a north or south aspect, subterraneous passages for conveying air being avoided. For artificially heating any enclosed space, the most healthful is by an open fire, or by direct radiating heating surfaces. In classing buildings in which the conditions for their ventilation and warming are various, the Author observes how difficult it is to fix for any one building a general principle as a basis of operation.

For the renewal or expulsion of the air by mechanical means,

water as a motive power is to be preferred to either steam or air, as giving off its foot-pounds of work with the least loss in its dynamical value. Comparing the transport of the units of heat through a building to the transport of materials, the Author gives preference to steam, as having a less "deadweight" than either air or water. To illustrate this, the Author says that to raise 1 kilogramme of air 1° C. will absorb 0·25 unit of heat (caloric). To raise 1 kilogramme of water 1° C. will require one unit of heat, while to raise 1 kilogramme of steam 1° C., as it already contains in latent heat 350 units of heat, requires only 0·305. Thus the "deadweight" to transport or convey over a building one kilogramme of each of these substances by any apparatus will be:—

$$\text{Air} \quad . \quad . \quad \frac{1 \text{ kilogramme}}{0 \cdot 25 \text{ unit of heat}} = 4000 \text{ grammes.}$$

$$\text{Water} \quad . \quad . \quad \frac{1 \text{ kilogramme}}{1 \text{ unit of heat}} = 1000 \quad ,,$$

$$\text{Steam} \quad . \quad . \quad \frac{1 \text{ kilogramme}}{350 \cdot 305} = 0 \cdot 00181 \quad ,,$$

In conclusion, steam as a heating medium has also the advantages of occupying less space, and gives off its units of heat more rapidly than either air or water.

W. W. P.

Cost of the Different Modes of Lighting the Town of Munich.

By F. Erismann.

(Bulletin de la Société d'Encouragement, 1878, p. 323.)

From a table, of which the following is an abstract, it appears that, of all the substances employed for lighting, petroleum is the most economical. But, taking everything into account, it is sensibly dearer, and the preference is given to gas for lighting.

——	Lighting Power, in Candles.	Consumption and Cost of Material in Twenty-four Hours, for a Power of Six Candles.	
			d.
Petroleum, with split burners . .	$10\frac{3}{4}$	17·8 oz.	2·52
,, ,, round ,, . .	8	25·6 ,,	3·36
Oil lamp	4	28·4 ,,	7·44
Coal gas	8	89·04 cubic feet.	6·57
Stearine candle	6	51·2 oz.	9·50

The superior performance of petroleum with the split burner compared with the round burner is noteworthy. Herr Erismann found that too much air was admitted to the round burner, and that the split burner gave a better light with less air.

D. K. C.

The Lighting of Towns. By M. Darcel.

(Annales des Ponts et Chaussées, 5th series, vol. xv., p. 449.)

This article furnishes a short treatise on the manufacture of coal gas. Reference is first made to the methods of lighting with colza oil and petroleum; the former is used in a few isolated districts of Paris where gas mains have not been laid down, but the light from these lamps is less powerful and more expensive than from gas lamps. The lamps supplied with mineral oil give nearly double the light of the colza oil lamps at the same cost, but the light is not so good as that given by gas. The Author then proceeds to describe the various well-known apparatus and processes employed in the production, purification, storage, and distribution of coal gas, and the burners in use. The apparatus of Pelouze and Andouin is used sometimes in France for removing tar from gas by causing the gas to impinge against a surface before being led into the purifiers. The gas is made to pass through a series of holes pierced in a triple coated cylinder, the holes being so arranged that the gas is forced to impinge twice against the sides, and the cylinder is so regulated, by the pressure of the incoming gas on a counterpoised cylinder to which it is connected, that it sinks or rises in an annular vessel containing water, exposing a smaller or greater number of holes according as the pressure of the gas is diminished or increased. The pressure at which the gas has to be introduced for the proper working of the apparatus renders it inapplicable for works not furnished with extractors, that is, producing less than 35,000,000 cubic feet of gas per annum.

The cost price of gas depends mainly on the price of coal, the value of the coke, the capital expended, and the total make of gas. Some details relative to the manufacture of gas in twelve gas works in France are given in a table. The regulations under which gas companies are placed in France vary considerably in different towns. Generally only one gas company is allowed to supply a town, and where two or more are admitted, different districts are assigned to them. In return for the privilege of laying their pipes along the streets, the gas companies are required to supply gas at a charge not exceeding a fixed limit, and of a definite quality. Frequently the gas companies are bound to supply gas for public lighting at half-price, but this regulation, though justified on the ground of their using the public roads under which to lay their pipes, increases the cost to private con-

sumers, and thus tends to restrict the consumption of gas to the detriment of the company. Some towns stipulate that they and the gas company shall divide any excess of dividend above 10 per cent., which arrangement works very satisfactorily. Concessions are generally granted for a term of years. Various decrees and public documents relating to gas manufacture are appended to the article.

L. V. H.

On Electric Lighting in Industrial Establishments.

(Journal für Gasbeleuchtung und Wasserversorgung, vol. xxi., p. 381.)

Serrin's lamp would not work in the foundry of Meer Brothers at Gladbach owing to dust fouling the delicate mechanism of the carbon holders. Herr Becker therefore constructed a lamp on the same principle to meet the inconvenience. In Serrin's system, however, when a piece of carbon breaks off, the lamp is out until the clockwork brings them again in contact. This is remedied by the Jaspeud regulator, but it cannot be kept free from dust. A lamp constructed on the principle of the beam balance has worked very well in Meer Brothers' foundry without any protecting cover. A comparison of the cost of lighting by electricity and by gas from actual data in each case gave 1 to 2·66 as result. The following are the areas lighted by one lamp at the different establishments. At Messrs. Muehlen and Co.'s spinning mills at Rheydt 412 square mètres, replacing forty-eight gas burners. At the spinning mills of Messrs. J. F. Clauser at Gladbach, 470 square mètres, replacing thirty gas burners. At Messrs. Swagemakers and Toonen, Tilburg, 440 square mètres, replacing forty burners. The hissing of the carbons, which was especially noticed when there was much ash, and was probably due to absorption of mineral substances in the carbons, might be eliminated by adding a flux which would bring the substances to their melting points. A medium-sized Siemens' machine gave a stronger current through Becker's lamp than the Gramme previously used. It was found possible to maintain two Becker's lamps in one circuit; and it seems now possible with this form of regulator to have a number of sources of light worked by the current from one machine.

F. J.

Electric Lighting. By H. Fontaine.

(Revue industrielle, vol. ix., pp. 222–224.)

For some time past a number of public places in Paris have been illuminated with the Jablochkoff electric candles, worked by a new type of Gramme machine, giving alternate currents. This machine is of the following description:—To a base plate are

bolted two standards of nearly circular form, which carry the bearings for the axis. These standards are fastened rigidly together by means of eight brass rods 580 millimètres (22·8 inches long. A hollow octagonal drum of cast iron is keyed at each end on a steel spindle 960 millimètres (37·7 inches) long, and revolves between the two standards. On each of the eight plane surfaces of the drum a flat core of soft iron, 270 millimètres (10·6 inches) long, is fixed. Each core is wound lengthwise with insulated copper wire, thus forming eight electro-magnets. The armatures of these magnets are of a curved form on the face; they extend on each side of the core so as to leave only a small space between each, and are supported at the ends by thin plates screwed to them and to the drum. Surrounding the electro-magnets, and in close proximity to the armatures, is a series of copper wire bobbins encircling either a hollow cylinder of soft iron, 320 millimètres (12·5 inches) long and 20 millimètres (0·78 inch) thickness of metal, or a number of curved segments of soft iron held together by a band of copper. This hollow cylinder is supported on flanges screwed to the brass rods above-mentioned, and within it the system of electro-magnets rotates. On the spindle of the machine are fixed a driving pulley and two insulated brass rollers; against the latter press brushes of silvered copper wire. A casing of polished mahogany encloses the apparatus; the bobbins are cooled by the passage of air through numerous holes in the casing. To start the machine a current of electricity from a continuous current Gramme machine is directed through the wire brushes into the electro-magnets, thereby magnetising the cores and armatures which revolve very close to the wire helices. Induced currents are generated in the latter of a strength depending on the magnetic force of the inductor and its speed of rotation. The poles of the electro-magnets being alternately north and south, the induced currents change direction at each moment. By the theory of induction the currents generated in the wire bobbins vary in intensity according to the position of the bobbins with respect to the armatures; the intensity is, however, constant in all the bobbins for one and the same position. Calling the bobbins *a*, *b*, *c*, *d*, each of the four bobbins comprising the *a* group is influenced in the same manner during a complete revolution of the electro-magnet, as also each of the four bobbins of the *b*, *c*, and *d* groups, whatever be the position of the armatures with regard to the entire number of helices. Consequently, for four tension circuits the eight *a* bobbins must be joined together; also the respective eights of the groups *b*, *c*, and *d*. The several helices of each group of bobbins are connected to exterior terminals. There are thirty-two bobbins, so that thirty-two distinct currents can be obtained, or the helices can be coupled up so as to give either sixteen, eight, or only four currents. At the present time three different sizes of these machines are made. The largest is 890 millimètres (35 inches) long, inclusive of driving pulley, 760 millimètres (30 inches) wide, and 780 millimètres (30·7 inches)

high; weight, 650 kilogrammes (1,433 lbs.); number of revolutions per minute, 600. When driven by a 16-HP. motor this machine feeds sixteen Jablochkoff candles, each equal to one hundred Carcel burners. The cost, including a small charging machine, is 10,000 francs. The medium size machine is for six candles of one hundred Carcel burners each; it absorbs 6 HP., and costs, with the exciting machine, 5,000 francs. The dimensions are: length, 700 millimètres, width, 400 millimètres, and height, 520 millimètres; weight, 280 kilogrammes (617 lbs.); number of revolutions, 700 per minute. The small machine sustains four candles, each of one hundred Carcel burners; it absorbs 4 HP., makes 800 revolutions per minute, and weighs 190 kilogrammes (418 lbs.). The dimensions are: length, 550 millimètres, width, 400 millimètres, and height, 480 millimètres. The cost, with charging machine, is 3,500 francs.

The Author also gives some details about the electric candles, invented by M. Jablochkoff to avoid the use of complicated regulators, or electric lamps, in which the positive carbon burns away twice as fast as the negative one when in circuit with a continuous current machine. With currents of alternate direction each carbon is consumed at the same rate. In the candles now used plaster takes the place of kaolin, the light being thereby doubled without increase of motive power. The candles are of better quality than formerly and are twice the length; their cost is less, and they last each one hour and a half. The present machines cost 10,000 francs, and each one feeds sixteen candles at 0·75 francs per candle, giving 0·50 francs per candle per hour, exclusive of motive power and interest for wear and tear and renewals. The old machines used for this system of lighting cost 7,000 francs, and each one could only sustain three candles. In a word, the progress made in this direction has been considerable; and although lighting by electric candles of equal intensity to the light of regulators is more costly, the former have the advantage of burning with surpassing regularity and softness, and are well adapted for the illumination of public gardens, theatres, &c.; but not so well for factories and workshops, where the question of first outlay is one of primary importance.

J. J. W.

On an Improvement in the Manganese Battery.

By M. Leclanché.

(Comptes rendus de l'Académie des Sciences, vol. lxxxvii., p. 389.)

In order to avoid the inconveniences attending the use of the battery presented to the Academy in 1876, and to render the resistance of the battery constant, the Author has sought to render this resistance independent of the conductivity of the mass of the compound and of the adherence of the electrode to this mass. To

this end the compound has been submitted to the hydraulic press, under the form of plates placed side by side with a plate of retort carbon, presenting a surface of about half a square decimètre. In this case the internal resistance of the battery is a function only of the conductivity of the exciting liquid, and this conductivity tends more to increase than to diminish, because the solution taking up chloride of zinc becomes better conducting. There is the further advantage that the depolarising plate can be easily replaced, when exhausted, by a new plate in connection with the carbon-plate. Small elements of low resistance, great power, and well adapted for the firing of electric fuses and general field-work can be constructed on this principle.

P. H.

New Insulating Support for Static Electricity.

By E. Mascart.

(Journal de Physique, 1878, p. 217.)

Sir W. Thomson has frequently insisted upon the necessity of exercising particular care in the insulation of apparatus for the study of static electricity, and in all his instruments provision is made for drying the supports by sulphuric acid. For the same purpose M. Mascart has devised a stand consisting of an annular hollow glass vessel, having the inner ring prolonged upwards so as to form a pedestal, while the top of the outer ring is contracted to a bottle neck, the clearance between the inner and the outer rings being reduced to 0·079 inch. The pedestal is surmounted by a brass cap provided with a screwed socket for carrying any form of plate. The space between the pedestal and the outer ring contains the acid which is introduced or withdrawn through a stoppered mouth. An india-rubber cap is drawn over the top of the stand when not in use.

F. J.

On Variations of Intensity of Electric Currents at bad Contacts.

By T. du Moncel.

(Comptes rendus de l'Académie des Sciences, vol. lxxxvii., p. 189.)

One of the most interesting methods of illustrating the variations of the intensity of currents transmitted across bad contacts, according to the pressure exerted, is to wind on a glass tube a helix of uncovered copper wire (say No. 16), and to adapt to the two ends of the tubes two systems of adjusting screws, so arranged as to compress the spiral when the screws are advanced towards each other, and thus pinch the convolutions more or less together. When the pressure is weak, the resistance of the spiral is only slightly less than would obtain if the wire were covered with silk, but as

the pressure is increased, the resistance is diminished, until the pressure attains its maximum. When the wire is clean, this effect is less marked than when it is slightly oxidised, but it is nevertheless clearly visible; and as the inverse effect is produced when the screws are released, it cannot be attributed to simple action of the layer of oxide that may cover the wire. The Author made this experiment in 1864, and the principle is now employed with advantage in certain cases, as in avoiding sparks from the extra current;[1] and he thinks that sufficient attention has not been given to the physical effects produced at the points of contact of bodies traversed by a current. There is actually a resistance to the passage of the current, which varies with the pressure exerted on the pieces in contact. Is the effect to be attributed to increase of the surface in contact, or to definite repulsions between the contiguous elements of the same current, more easily effected with slight than with strong contact, or to molecular vibrations? The point is worthy the attention of investigators, but it is certain that the effect is more characterised when the resistances to passage are considerable, and the number of contacts large.

P. H.

On Increase of Length in Electrical Conductors. By R. Blondlot.

(Comptes rendus de l'Académie des Sciences, vol. lxxxvii., p. 206.)

A conductor traversed by a current is heated, and consequently lengthened. Besides this easily demonstrated effect, there is said to exist a dilatation, produced directly by the current by its mechanical action. The experimental solution of this problem presents great difficulties because of the co-existence of the thermic dilatation and of the effect sought, if it exist. Edlund and Streintz concluded that there existed a purely electrical lengthening. Wiedemann regarded these experiments as insufficient, and the question as unresolved. The Author has designed an entirely different method of experimentation, from which he determines that the passage of a current in a metallic conductor produces no mechanical increase or decrease of length.

P. H.

A Theoretical Deduction of the best Resistance of a Telegraph Receiving Instrument. By R. S. Brough, Assoc. Inst. C.E.

(Proceedings of the Asiatic Society of Bengal, 1877, pp. 184–188.)

The Author remarks that the general rule given in text-books for the resistance of a telegraph receiving instrument is not appli

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[1] *Vide* Annales télégraphiques, vol. viii., p. 211, for March–April, 1865.

cable to high speed signalling and very long and highly insulated lines. The best resistance for an electro-magnet to be employed on any line must be considered from two aspects, which may be termed the "static" and "kinetic." The first aspect involves the problem, to find the resistance of the receiving instrument which will make its magnetic force a maximum when a steady current, that is, one which does not vary in strength with respect to time, is flowing from the sending to the receiving station, and this is completely solved.

Let r = resistance of receiving instrument, f = resistance of battery, k = resistance of conduction per unit of length, i = resistance of insulation per unit of length, and l = length of line. Then it can be shown (Blavier, "Annales Télégraphiques," 1858, p. 234), that the magnetic force is a maximum for

$$r = \sqrt{ki}\left\{\frac{\sqrt{ki}\left(1 - \epsilon^{-2l\sqrt{\frac{k}{i}}}\right) + f\left(1 + \epsilon^{-2l\sqrt{\frac{k}{i}}}\right)}{\sqrt{ki}\left(1 + \epsilon^{-2l\sqrt{\frac{k}{i}}}\right) + f\left(1 - \epsilon^{-2l\sqrt{\frac{k}{i}}}\right)}\right\}.$$

Let A = measured insulation of line with distant end insulated, and B = measured conduction of line with distant end to earth. Then in the above—

$$k = \frac{\sqrt{AB}}{2l}\log_\epsilon \frac{\sqrt{A} + \sqrt{B}}{\sqrt{A} - \sqrt{B}} \text{ and } i = \frac{AB}{k}.$$

If the resistance of the battery f may be neglected—

$$r = \sqrt{ki}\,\frac{1 - \epsilon^{-2l\sqrt{\frac{k}{i}}}}{1 + \epsilon^{-2l\sqrt{\frac{k}{i}}}}$$

= measured resistance of line with distant end to earth.

From this value of r a considerable reduction has to be made on account of the thickness of the insulating covering of the wire in the receiving instrument according to the formula.[1]

$$\frac{\text{Resistance of receiving instrument}}{\text{External resistance.}} = \frac{\text{Diameter of bare wire}}{\text{Diameter of covered wire.}}$$

The "kinetic" problem, in which the influence of the resistance of the instrument on the speed of signalling is considered, has never been completely solved.

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[1] *Vide* Proceedings of the Asiatic Society of Bengal, 1877, p. 134.

Sir William Thomson has shown that the speed of signalling on any line depends on the value for that line of a certain constant termed the "retardation characteristic" of the line. Let k = the resistance, and c = the capacity of the line per mile, and l = length of line in miles, then the expression for this constant is

$$\mathrm{R\,C} = \frac{k\,c\,l^2}{\pi^2} \log \epsilon\,(\tfrac{4}{3}).$$

Thus the value of the RC increases as the square of the length of the line; now by increasing the resistance of the receiving instrument the length of the line is virtually increased, and if the resistance of the instrument be made unduly high, the value of the RC may be increased to such an extent as to impair the signalling speed of the line. It is clear, therefore, that in the case of a very long and highly insulated line the best resistance for the receiving instrument, as given by the "statical" formula only, may be so great as to retard the speed of signalling.

For a perfectly insulated line, let l = length of line in miles, k = resistance per mile in ohms (supposed uniform), c = capacity per mile in farads (also supposed uniform), and r = resistance in ohms of the receiving instrument. Then the sensibility of the receiving instrument is

$$\mathrm{M} = \text{const.} \times \frac{\sqrt{r}}{r + k\,l}.$$

Assuming that the insertion of the receiving instrument of resistance r in the circuit has approximately the same influence on the signalling speed as increasing the length of the line by $\frac{r}{k}$ miles, then

$$\mathrm{R\,C} = \text{const.} \times \frac{k\,c\left(l + \frac{r}{k}\right)^2}{\pi^2} \log \epsilon\,(\tfrac{4}{3}).$$

Now, if it may be assumed that the efficiency of the receiving instrument varies directly as its sensibility, but inversely as its retardative influence, then the expression for the efficiency is

$$\mathrm{R\,E} = \text{const.} \times \frac{\pi^2 \sqrt{r}}{k\,c\left(l + \frac{r}{k}\right)^2 (r + k\,l) \log \epsilon\,(\tfrac{4}{3})}$$

$$= \text{const.} \times \frac{\sqrt{r}}{(r + k\,l)^3},$$

which is a maximum for

$$r = \frac{k\,l}{5},$$

showing that for a perfectly insulated and uniform line the resistance of the receiving instrument should be one-fifth of that of the line, which is the precise value selected on experimental grounds by Prof. Hughes.

Introducing the resistance of the signalling battery, consisting of a certain number of cells, all of equal electro-motive force and resistance, modifies the result.

Let the cells be arranged so that the total resistance of the battery $= f$, then it may easily be shown that the total electromotive force of the battery will be proportional to $\sqrt{f}$, so that using the same notation as before, the expression for the sensibility of the receiving instrument is

$$M = \text{const.} \times \frac{\sqrt{f\,r}}{f + r + k\,l},$$

and for the retardation characteristic—

$$R\,C = \text{const.} \times \frac{k\,c\left(l + \frac{f+r}{k}\right)}{\pi^2} \log \epsilon\,(\tfrac{1}{3}).$$

The receiving efficiency of the instrument is expressed by

$$R\,E = \text{const.} \times \frac{\sqrt{f\,r}}{(f + r + k\,l)^3},$$

which has a maximum for f and r, viz.:—

$$\left.\begin{aligned} r &= \tfrac{1}{5}\,(f + k\,l) \\ f &= \tfrac{1}{5}\,(r + k\,l) \end{aligned}\right\}.$$

These maxima conditions are simultaneously fulfilled by

$$r = f = \tfrac{1}{4}\,kl.$$

J. J. W.

Report on the Use of Metal Telegraph Posts.

By F. Evrard.

(Annales des Travaux Publics de Belgique, vol. xxxv., p. 5.)

The Telegraphic Administration having instructed the Author to study the different types of metal telegraph posts or poles used in France, he has thought it advantageous to include the different types employed in other countries. The following classification has been adopted: cast-iron posts; iron posts, including posts of angle, cylindrical, conical, T and double T iron; sheet-iron posts; composite posts.

Cast-iron Posts.—In Prussia there was employed, as well as the angle-iron post, a strong column 7·39 mètres (24·2 feet) in length, and weighing 2,464 lbs., resting on a plate weighing 979 lbs., buried 1·67 mètres (5½ feet) in the earth. The price, 480 francs, was too high for the use to be continued.

Angle-iron Posts.—The Swiss administration constructed in 1857 the lines from Laufelfingen to Sissach, and from Hauenstein to Olten, with angle-iron posts, but it soon appeared that these posts were too weak and too short.

Cylindrical Posts.—In 1858 the same administration adopted another system; each post was formed of two or more cylindrical tubes tapering towards the top, which were connected by an iron ring. This post has received the name of the "muffle" post; 208 kilomètres of line were constructed with it, but at the end of three years it was found that the points of junction presented insufficient stability. In Prussia, on the line from Weissenfels to Gera, tubes of wrought-iron gas-pipe of 0·008 mètre to 0·078 mètre diameter were tried, filled with cement to give increased resistance, but these were considerably damaged by storms, and the number of wires carried was too small. They have been replaced by injected wood posts.

Conical Posts.—In 1861 the Swiss Administration devised a conical post in a single piece which appeared to fulfil all desirable conditions. This post was employed in the construction of 590 kilomètres of line, but wood posts have been maintained on railways in levels, where the iron posts were of insufficient length. The length varied in twelve sizes from 2·50–3 to 5·28 mètres; the smaller sizes were too short, and the largest offered insufficient resistance to the traction of the wires. These posts were found to be more durable than injected posts, but this advantage was counterbalanced by serious objections as to price, maintenance, strength, and number of wires carried. The Swiss administration have concluded that stronger and higher posts were required; but it would be difficult to obtain in a single piece conical posts 7 to 8 mètres in height, and the price would be too high.

T-Iron Posts.—M. de la Taille, telegraph inspector at Orleans, has invented a T-iron post furnished with horizontal bars, of 4 feet length in square iron of 1 inch side, and 15¾ inches apart, to carry four insulators, also 15¾ inches apart. The post is steadied in a block of concrete varying from 50–200 decimètres cube (1·7 to 6·8 cubic feet), according to the height of the post and the number of wires. The use of the concrete appears to present the double advantage of preserving the buried part of the post from oxidation, and of ensuring its stability. The iron should support a traction of 28 kilogrammes per square millimètre without breaking, and elongate under increasing weights 6 per cent. On the line running with the Orleans Railway, the posts on this system are 7·60 to 9 mètres in length, the lowest wire being 4 to 5·40 mètres above the ground, and weigh 16 kilogrammes per mètre. Seventeen wires are carried.

Double ⊤ *Iron.*—When simple ⊤ iron is used for lines of more than ten wires, with four-wire traverses, it is necessary to adopt a section such that the weight of the lineal mètre be above 11 kilogrammes, otherwise there is not sufficient resistance in the direction perpendicular to the wires. This resistance can be obtained with double ⊤ iron of only 7·50 kilogrammes per lineal mètre (15 lbs. per yard). On the line from Munich to Augsburg, these posts carry sixteen wires of 4·5 millimètres diameter for a distance of 62 kilomètres, and weigh 16·50 kilogrammes per mètre. A somewhat similar system is adopted on the Bavarian lines, but with the exception that angle iron is used instead of square iron for the cross arms, and a granite block to receive the post, costing, with the necessary boring, five times more than the concrete block.

The Use of Rails as Posts.—In the Grand Duchy of Mecklenburg rails are used for posts, and those erected in 1862, 1865, 1867, still exist in good preservation. The rails are of 5·63 mètres (18·4 feet) length; when required to be longer, two rails are coupled together. The posts are set 1·25 mètre in the earth, where practicable in the rock.

Sheet-Iron Posts.—There are employed in Prussia, at angles and to resist exceptional strains, sheet-iron columns formed of four plates connected at the edges by rivets; the space enclosed forms at the top a square of 0·175 mètre in the side, and at the bottom of 0·420 mètre. The post is buried 2·60 mètres in the earth; its height is 8·60 mètres (28·2 feet); it weighs 560 to 600 kilogrammes, and costs 400–500 francs. M. Desgoffe constructs a post of two plates bent to a curve, offering great resistance, and riveted at the edges. Where ærial and subterranean lines meet, these posts have been used at the juncture, the cable being brought up the hollow post. The Desgoffe posts have been substituted in the neighbourhood of Paris for wooden posts when it has become necessary to raise the wires. M. Papin has designed a post formed of four sheets of iron connected in a square by four angle irons. The great inconvenience attending their use is that the buried portion has very little durability, and it has been sought to remedy this by planting the post in Portland cement.

Composite Posts.—The line from Paris to St. Germain, constructed in 1865, is entirely composed of posts of cast and malleable iron, made in two parts. The lower is a cast-iron column of 3·50 to 4·50 mètres (11½ to 15 feet) height; it is terminated at the summit by a ring of iron of 0·10 mètre height, and 0·07 mètre diameter. In this ring is set a small post of + section, 2·50 mètres (8·2 feet) in height. The posts are set in masonry. On this line they are in good preservation, and carry six to nine wires. This type could not be employed on a great line, for it would become necessary to increase the section of the cross iron, and this would entail considerable expense. The Siemens is also a composite post formed of two tubes, surmounted by a bar of 0·50 mètre height, which serves as a lightning conductor. The base is a plate curved or buckled

in the centre where it is riveted to the cast-iron tube, and the latter carries the upper malleable iron tube set in a cement. As many as forty thousand of these posts are in use on the Indo-European line alone, and their use extends to many countries, but the Author observes, that although they may have an advantage on lines established in countries where superintendence and maintenance are difficult, this is not the case in France where the conditions are very different; and the price of these posts is very high. The "Riband" post consists of a cast-iron base surmounted with iron ribands wound helically and running to the head of the post. These ribands are connected by two bands throughout the length of the post, and at the points where the ribands cross they are riveted. The post is elegant in appearance, and adapted for use in towns. M. Papin has designed a post consisting of four angle irons connected by crossed bands. The price of these posts is too high for construction in the run. M. Desgoffe, guided by the French Administration, has modified the type of his post. To diminish oxidation in the earth he has adopted a cast-iron base formed of two conical pieces grasping the sheet iron for a certain length and bolted together.

Choice of the best Systems.—The Author points out that the angle iron, cylindrical and conical posts, present so many inconveniences that in Germany and Switzerland the trials have been discontinued. The posts composed entirely of sheet iron are not of sufficient durability. The "Riband" post is too high in price to be employed in current construction. The Siemens post appears to the Author also too costly, and to be convenient only in countries where superintendence and maintenance are difficult. This leaves for consideration in a more detailed manner the posts in T and double T iron, and the composite post on the Desgoffe system.

Maintenance and Service.—The loading and unloading of iron posts demands greater care than for those of wood; the iron may become bent or broken, and the concrete blocks fractured. The weight for transport will be considerable if the posts are in T or in double T iron; the Desgoffe posts are not heavier than wooden posts. The planting is as easy as with wooden posts. Replacement of posts in T iron will be more difficult than that of wood posts, and the socket will often be lost. The Desgoffe posts must be painted every two or three years, the posts in T iron will not need so much attention. Metal posts need renewal less frequently than wood posts. A post having small surface, compact, sufficiently stout, without rivets or bolts, will be more durable than any other, and the T-iron form best answers to this condition. The larger diameter of the Desgoffe posts, the larger surface and easy corrosion, tend naturally to more rapid destruction of the lines. The general objection to the use of iron posts is the first cost; and in the majority of cases wooden posts are preferable from an economical point of view. The Author enters at some length into the subject of relative costs.

General Considerations.—Iron posts have not been definitely

adopted by telegraph administrations; some countries have given up the trial. Only posts in simple T iron appear to compete with wood posts. The life of a wooden post is about twenty years, and it is only necessary that they should be of sufficient height to bear the number of wires to be provided during this period, but this is not the case for iron posts where the principal advantage is very long life. It is therefore necessary that posts be chosen in such conditions that they will carry supplementary wires as the increase of traffic may necessitate. As to the duration of an iron post this will vary with the metal, the form adopted, its dimensions, the earth in which it is set (if there is no socket); its life may be lengthened by careful maintenance, but rust will always exercise destructive effect.

A good type of iron post should fulfil the following conditions: It should have sufficient resistance. Angle posts must be specially constructed. A long life should be looked for, which needs that the post should not easily be attacked by rust, nor by the destructive agencies of climate and soil. After setting it should have great stability, and socket posts best satisfy this condition. The setting of the post should be simple, and transport easy. The price should not be high. The simpler types answer the latter conditions.

P. H.

The Microtasimeter.

(Comptes rendus de l'Académie des Sciences, vol. lxxxvii., p. 269.)

Mr. Edison presented to the Academy, through M. du Moncel, a model of his invention, the microtasimeter, an instrument intended to measure infinitesimal differences of temperature or of humidity. This apparatus is based, like the carbon-telephone, upon the principle of variation to which a current is subject when it passes across two juxtaposed bodies, the pressure between these being made to vary. It consists of a rigid system, to which is adapted a carbon disc, interposed between two plates of platinum, and against these bears a solid arm, arranged so as to receive the action of a bar sensitive to the variations of heat or of humidity. This bar is arranged horizontally, and is supported at the opposite side to that which acts upon the discs by a thumbscrew, which admits of regulating the initial pressure exerted. The two discs of platinum between which the disc of carbon is enclosed are connected with the two branches of a Wheatstone bridge circuit, and the greater or less pressure exerted on the carbon by the horizontal bar, when it dilates or contracts, cause considerable variations of resistance in the corresponding branch of the bridge, which variations may be exactly measured, and indicate consequently the lengthening or shortening of the bar, however small this may be. It is necessary that the bar be short, and present only a small surface, and that the carbon be prepared in a particular manner.

Smoke-black, produced by petroleum lamps, and slightly compressed, gives the best effects, and Mr. Edison has found, among the substances used for the bar, that ebonite is most favourable to calorific effect, and gelatine to hygrometric effect.

P. H.

On the relation of Work done by Diffusion to the Second Law of Thermodynamics. By R. CLAUSIUS, Hon. M. Inst. C.E.

(Annalen der Physik und Chemie, new series, vol. iv., p. 341.)

Mr. S. T. Preston has described[1] an apparatus by which mechanical work can be obtained simply from diffusion of gases, and which he demonstrates as contradicting the second law of Thermodynamics. The principle of the apparatus is as follows: A cylinder has a porous diaphragm separating two gases, e.g., hydrogen and oxygen; the former will pass through the diaphragm quicker than the oxygen, and thus a greater pressure will exist on the oxygen side, and the diaphragm may be caused to move; now the gas will expand on the one side and be compressed on the other, which causes a transference of heat from a colder to a warmer body. The Author shows that were the gases at the end of the experiment in the same condition as at the commencement Mr. Preston would be right, but the gases are mixed at the close, and the greater disgregation of the molecules is equivalent to a positive increase of heat, which is balanced by the heat turned into work and transferred to a warmer body, both of which are negative quantities. Therefore this law not only holds good but is corroborated by the case of diffusion of gases.

F. J.

On the Dependence of the Specific Heat at Constant Volume and the Conductivity of Gases on the Temperature. By A. WÜLLNER.

(Annalen der Physik und Chemie, new series, vol. iv., p. 321.)

Comparing the experiments of Stephan, Winkelmann, Kundt, and Warburg, and Plank on the conductivity of gases, with the theoretical equation $K = 1 \cdot 53\, \eta\, c$ (η being the co-efficient of friction, and c the specific heat at constant volume), the Author found accordance only for the diatomic gases, and not for the higher atomic, and he deemed this due to the alteration of the specific heat and frictional co-efficient with the temperature.

The Author experimentally determines the velocity of sound in the various gases by measuring the wave-length, and number of

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[1] *Vide* "Nature," vol. xvii., p. 202.

vibrations produced in a glass tube filled with the dried gas; and then by the known equation obtains the ratio of the specific heats at constant volume and pressure. Very great care was exercised, as is evinced by the detailed account, and the mean of totally separate experiments taken as the result.

The following are the results taking the specific heats at constant pressure as determined by Regnault and Wiedemann (marked R. or W. in columns II. and V.).

—	At 0° Centigrade.			At 100° Centigrade.		
	Sp.H. pressure / Sp.H. volume	Sp. H. at constant pressure.	Sp. H. at constant volume.	Sp.H. pressure / Sp.H. volume	Sp. H. at constant pressure.	Sp. H. at constant volume.
Dry air	1·40526	0·23751 R.	0·16902	1·40289	0·23751 R.	0·16930
CO	1·40320	0·2426 W.	0·17289	1·39465	0·2426 W.	0·17395
CO_2	1·31131	0·1870 R.	0·14260	1·28212	0·2145 R.	0·16730
		0·1952 W.	0·14886		0·2169 W.	0·16917
N_2O	1·31060	0·1983 W.	0·15130	1·27238	0·2212 W.	0·17384
Ethylene	1·24548	0·3364 W.	0·27007	1·1870	0·4189 W.	0·35366
Ammonia	1·31720	0·5009 W.	0·38026	1·2770	0·5317 W.	0·41685

The Author finds his values agree with those of Rontgen when reduced to the same temperature.

He next calculates the thermal conductivity with the values of η determined by Obermaier for 0° and 100° C., and the above values for Sp. H. at constant volume.

Comparing these with the observed results of Winkelmann the following table gives the results:—

—	Thermal Conductivity				$\frac{K_{100}}{K_0}$.	
	At 0° = K_0.		At 100° = K_{100}.			
	Calculated.	Observed.	Calculated.	Observed.	Calculated.	Observed.
Air	0·0000434	0·0000513	0·0000553	0·0000653	1·2747	1·2770
CO	430	499	545	..	1·2674	..
CO_2	315	305	476	466	1·5106	1·5300
N_2O	313	350	483	506	1·5418	1·4468
Ethylene	381	395	673	636	1·7668	1·6110
Ammonia	..	458	..	709	..	1·5475

If for air η_o from O. E. Meyer's determination is taken K_o becomes 0·0000469, which is more accordant with the theoretical value.

F. J.

The Singing Telephone at the Stevens Institute.

By H. MORTON, Ph. D.

(Journal of the Franklin Institute, August 1878, p. 112.)

A number of experiments have been made at the Stevens Institute of Technology, by Messrs. W. E. Geyer, H. A. Beckmeyer, and B. Ayres, with the object of developing a loud-sounding instrument to transmit musical tones. As the result of numerous experiments, the transmitter devised by Mr. Geyer was found to give the best results. An ordinary U-shaped electro-magnet, about 3 inches long, wound with coarse wire, so as to have only about two ohms resistance, was supported with its limbs vertical, the poles upwards. A narrow thread of soft india-rubber was laid on each pole, and upon these threads there rested an ordinary armature cemented to the resonant case of an Æolian harp. Vibrations were produced by fluctuations in the attractive force caused by the intermittent current; and the sound produced could have been heard over a large room. The magnet was, subsequently, placed in a horizontal position, and a guitar, carrying the armature, was suspended vertically against it. By the substitution of thick letter paper for the india-rubber, the range and certainty of action of the apparatus were greatly increased. A convenient battery, designed by Dr. Morton, consisted of ten glass tubes 1¼ inch by 4 inches, set in wood, with elements of carbon and zinc, excited by a strong solution of Glauber salt, or sal-ammoniac.

D. K. C.

The Manufacture of Swedish Matches. By Prof. M. SCHOENFLIESS.

(Zeitschrift des Vereines Deutscher Ingenieure, vol. xxi., p. 177.)

The Swedish matches differ from the common description in their igniting compound, which contains no phosphorus, and in the manner of transmitting ignition to the wood, for which purpose they are impregnated with paraffin, the use of sulphur being entirely dispensed with. The most suitable description of wood is aspen, in logs of 12 to 22 inches diameter, fine grained pine wood being, however, likewise used when aspen wood is scarce. In either case, the wood must be worked as soon as possible after being felled; if seasoned, it has to be steeped in water. The logs are cut by a cross-cut saw into pieces of 14 inches, each containing seven lengths of matches, the bark being removed immediately afterwards by hand labour. The pieces are next chucked in a spear lathe, making 15–20 revolutions per minute, where they are reduced to shavings by a planing tool acting simultaneously over the whole length. The thickness of these shavings is equal to the required thickness of the matches. Fixed to the same rest as the planing

tool, but slightly above it, are eight cutters, which divide the shaving into seven equal breadths, whereof each corresponds to the length of one match.

These shavings are freed from knots, and cut into lengths of about 6 feet, from which the matches are produced by a machine similar to a guillotine paper-cutter. This machine operates upon two packs at once, each consisting of ninety shavings, and, if properly fed and making 120 strokes per minute, cuts matches at the rate of a million per working hour.

At this stage the match slips are dried, by being passed through two wire-gauze cylinders of about 10 feet in length, and 30 inches diameter, making 30 revolutions a minute, and placed one above the other within a brick stove heated by chips and waste. The dried slips are next freed from splinters by being placed on a grid, with openings of suitable width to effect the separation. This grid receives a rapid vibratory motion in a direction across its openings by a crank shaft. Its surface is partitioned by strips of zinc in the same direction into compartments a little wider than the length of a match, so that the slips are not only freed from splinters by rubbing against each other and against the bars of the grid, but are also laid parallel in these compartments.

The slips are next "dipped," for which purpose they have to be inserted into a frame consisting of 44 wooden laths, of 1 inch × ¼ inch section, and 28 inches long, having a hole at each end, and moving freely upon two round iron bars, fixed in a somewhat stronger lath. Between each two of these laths fifty cuttings have to be ranged, at equal distances from each other, and projecting equally beyond the surface of the frame. This is done by means of a "filling machine," consisting mainly of a cast-iron table, fitted with fifty parallel grooves of a depth equal to the intended projection of the cuttings beyond the surface of the frame. Upon this table the frame is laid, the laths being close to each other, and at right angles to the grooves. The laths are now shifted from each other to the distance necessary to receive the cuttings, by two systems of wedges, applied successively from each side between the laths (each two opposite wedges acting simultaneously), after which the frame is covered by a gridiron, swivelling on two studs, the openings of the same corresponding exactly with the grooves of the cast-iron plate. In consequence of these openings being at a right angle to the interstices between the laths, a kind of sieve is formed, each hole of it receiving a match slip. These are now filled in from a frame in which they have been loosely packed together, the bottom of which is fitted with a number of conical tubes, the lower end of each tube being the narrow one, and corresponding with a hole of the sieve before mentioned. This frame is laid on the top of the gridiron, and, in consequence of a slight vibratory motion being imparted to it, a slip drops into each hole. As soon as this is done, the slips remaining in the filling frame are shut off by a slide, the filling frame is removed, the laths of the frame (between the gridiron and the bottom plate) are set tight, so as to

retain the cuttings; lastly, the filled wooden frame is removed and another inserted.

The filled frames are next dipped into three heated pans, the first one being empty, and serving to heat the slips and rarefy the air within them; the second is filled with paraffin, and the third contains the igniting compound. The latter is sometimes laid on by means of a roller covered with india-rubber and fitted with a "doctor," by means of which a workman may finish eight millions of matches per diem.

After this the matches are brought to the drying-room, and on leaving it are removed from the frames and filled into boxes—either by hand, or by a separate "discharging machine," which is in many points similar to the filling machine. The wooden laths having been loosened at first, are forced asunder by two systems of wedges, as described before, after which the matches are detached from the latter by the action of a brush passing underneath them, and drop down upon the surface of a number of endless and parallel india-rubber bands. The distance of this surface from the bottom of the wooden frame is such, that, if the bands did not move, the matches would just stand upright; the consequence is, that all the matches fall down upon the moving bands in the same direction. Each band conveys the matches to a separate shoot, from which they drop into a large box, which when filled is taken out of the machine, and emptied by hand into the little boxes which are offered for sale.

A. H.

The Recoil of Cannons and Motion of Projectiles.

By M. Sebert.

(Bulletin de la Société d'Encouragement, 1878, p. 282.)

M. Sebert has lately brought under the notice of the Société d'Encouragement an apparatus, devised by him for the determination of the laws governing the recoil motion of guns, and the velocity of the projectile. This apparatus he calls a "velocimeter." It will, he says, determine with wonderful accuracy the distance traversed by the gun in recoiling during an interval of time as small as the $\frac{1}{1500}$ part of a second.

The determination of the law of recoil during the moments following the combustion of the powder in the gun is a question of great interest, inasmuch as, under given conditions, the pressure exerted in the bore may thereby be ascertained, and consequently the thickness of the metal be more accurately determined, and a powder be chosen which will give the greatest initial velocity combined with the least possible strain on the gun.

From the results of various experiments carried on by the Author, he has ascertained the fact, previously pointed out by the commission of Gavre, that the velocity of the carriage in recoiling

continues to increase perceptibly after the projectile has left the muzzle of the gun, an effect which the Author ascribes to the expansion of the gas still remaining in the bore. As an instance of this, he says that a 24-centimètre cannon, with a charge of 28 kilogrammes of powder, gave to a projectile weighing 144 kilogrammes an initial velocity of about 450 mètres per second; and he observed that the gun with its carriage had recoiled about 30 millimètres at the instant when the projectile left the gun; that is to say, at the end of 0·0114 of a second, the velocity of the recoil being at that moment 3·80 mètres per second. This velocity increased to a maximum of 5·20 mètres per second, which was attained when the gun had recoiled about 0·20 mètre, and when the projectile was already more than 15 mètres from the muzzle.

The indications of the apparatus are so delicate that the curve of velocity shows even the undulatory nature of the movement given to the recoiling mass, consequent, no doubt, upon the elasticity of the parts of which it is made up.

With suitable modifications, the velocimeter may be made to register the precise moment when the projectile issues from the mouth of the gun, or passes other points at given distances from it. It will register with the utmost accuracy the space passed through by the projectile in the $\frac{1}{100000}$ part of a second.

In the experiment made with the 24-centimètre gun, alluded to above, the following results were obtained:—Time occupied by the projectile in traversing the length of the bore, 0·01124 of a second. Time occupied in passing from the muzzle of the gun to a screen placed at a distance of 33 mètres, 0·07305 of a second. Time occupied in passing from this screen to another placed 83 mètres from the gun, 0·1127 of a second. From these results the Author calculated that the velocity of the projectile at 16 mètres from the gun was 451·2 mètres per second, and at 78 mètres it had fallen to 443·2 mètres per second.

G. G. A.

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LONDON: PRINTED BY WILLIAM CLOWES AND SONS, STAMFORD STREET AND CHARING CROSS.